Structural Analysis

Planning Assistance to States
Section 22, WRDA 1974, as amended

Union Ship Canal
Buffalo, New York

U.S. Army Corps of Engineers
Buffalo District

November 2002

Executive Summary

This report was prepared under Section 22, of the Water Resources Development Act (WRDA) 1974, as amended, which allows the Corps of Engineers to provide technical assistance to support state preparation of comprehensive water and related land resources development plans, including watershed and ecosystem planning. Section 22 is also known as Planning Assistance to States (PAS). Assistance is given on the basis of state requests and availability of Corps expertise rather than Congressional study authorization procedures. Cost sharing is based on a 50% Federal and 50% non-Federal basis. The investigation to determine the structural integrity of the Union Ship Canal and report on other factors that would be considered by the City in plans for future development, was undertaken in response to a request from Erie County and the City of Buffalo. All work performed under the Section 22 Authority, was identified in an Agreement, which was executed between the Corps of Engineers and the City of Buffalo on 15 July 2001. Several interrelated investigations have been incorporated into this report, these include a structural dive inspection of the canal walls, search pattern dives to determine existing bottom conditions, location of abandoned cars and debris; drilling, boring and exploratory excavations; geotechnical and stability analyses; sediment analyses, and a cost estimate for four alternatives.

The purpose of the sediment analysis was to determine if sediment from within the Union Ship Canal would be suitable for open lake disposal. The sediment was analyzed as directed in the Dredged Material Testing and Evaluation Manual (USEPA/USACE 1998). Samples were analyzed to determine if the following contaminants existed: Polychlorinated Biphenyls (PCB's), Polynuclear Aromatic Hydrocarbons (PAH's), Metals, Oil and Grease, and Ammonia Nitrogen. Results obtained from the testing were compared to an open lake reference area. Of the eleven sites tested, all had multiple PAH compounds that exceeded the open lake reference area levels, and the Toxicity Equivalent Factor model. Two PCB's were found, Aroclor 1254 and 1260. Heavy metals were detected in the samples, with Lead and Zinc found in particularly high levels. For sites 3 – 11, the lead levels exceeded the 100 ppm hazardous waste regulatory limit. Further Toxicity Characteristic Leaching Procedure (TCLP) testing is required to determine a suitable method and location for disposal of the sediments. Due to the high levels of contamination, open lake disposal is not an option. Disposal will require confinement in a confined disposal facility (Buffalo Harbor CDF#4 is nearby) or at an upland municipal landfill. A specially permitted landfill would be required if the TCLP test determines that the sediments are indeed a hazardous waste.

The structural dive inspection determined that the canal walls were timber crib structures supporting concrete caps. Both the timber cribbing and the profile of the concrete caps varied in design at various locations around the canal, and most notably on the north side. In general, the dive inspection noted that the cribbing appeared tight and in good condition. The timber cribbing should remain in good condition as long as it is continuously submerged. Recent low lake levels however have periodically exposed the upper 1' of the cribbing, which could subject this portion to deterioration. Two sections of the north wall show evidence of rotation; these sections are approximately 200 ft and 90 ft in length. Misalignment of the 200 ft section is estimated at up to 4 ft and the 90 ft section is 1 ft – 2 ft. Minor undermining of the south wall has also occurred, however is not expected to grow and is not currently a problem. A separate

dive inspection determined the existing bottom conditions and catalogued several vehicles and other debris present in the canal. The general condition of the concrete caps observed above water was good, with only minor cracking, spauling, and efflorescence.

The geotechnical investigation involved testing of concrete cores, determination of soil strength parameters to be used in the stability analyses, and exploratory investigations to determine structure types and dimensions. Soil samples and excavations were made at each of the reaches that could visually be determined by variations in the structure geometry. Structural stability analyses were performed using soil strength parameters determined by the geotechnical analysis. No historic or recent project drawings of Union Ship Canal were available, so all structural dimensions were determined by direct measurement during the exploratory excavations mentioned previously. Due to lack of rock strength data, simple overturning and sliding parameters were applied. The concrete cap at each reach was analyzed individually, with the assumption that it was rigidly connected to the timber cribbing. The stability analyses coincided with observations from the structural dive inspection, in that it was the northeast end of the canal that did not meet stability criteria. This was the same location that showed signs of distress and previous repair attempts. It is likely that the failures observed in these locations will be progressive and continue if no stabilization methods are undertaken. Four options for stabilization were formulated, these include; sand or rock berm stabilization, stabilization by partially filling the canal with fill material, and partially filling the canal to include a sheet pile wall across the width. Each estimate includes costs for concrete sidewalks and an asphalt bike path around the canal, railings for public safety, and a 25% contingency due to the preliminary nature of this investigation.

Stabilization Alternatives & Costs

Stone Slope Stabilization	$4,381,420
Sand Slope Stabilization	$2,839,912
Stone Fill to El. 577	$8,128,863
Fill and Sheet Pile Wall	$4,394,060

The recommended alternative is sand slope stabilization. Not only was this determined to be the most economical alternative, it would also best preserve the historical nature of the site as it would not be visible above water.

Conclusion

The Union Ship Canal was found to be a highly contaminated environment, particularly due to high lead levels. The canal has been used for dumping of various vehicles, debris and general trash. Structurally, it is in good condition with the exception of approximately 1400 linear feet of wall, which is showing distress in the form of bowing and rotation. It is recommended that this portion of the wall be stabilized using a sand berm to economically provide the needed stability while minimizing impact on the historical value of the site. The cost of this stabilization is estimated at approximately $2,840,000. Handrails, concrete sidewalks, and an asphalt bike path will be required for public use and safety.

US Army Corps
of Engineers
Buffalo District

Union Ship Canal
Buffalo, New York

Dive Inspection Report

Appendix A

January 2002

CELRB-ED-DC

January 2002
Chader x4188

Union Ship Canal, Buffalo, NY
Dive Inspection Report

Introduction

The Union Ship Canal is located along the Lake Erie waterfront near the southern limits of the City of Buffalo, New York. The canal was an integral part of the steel industry in the early part of the century, it measures approximately 1901 ft east/west and 200 ft north/south (Figure 1). The former Father Baker Bridge allowed vehicular traffic to pass over the canal while ships passed into and out of the canal to load and unload. As the steel industry slowed, and the deteriorated Father Baker Bridge was replaced with a much lower bridge, the canal was rendered inactive. The canal and surrounding area is currently slated for restoration and future development. In order to facilitate future plans, it was necessary to evaluate the integrity of the existing bulkhead structures. This included a bathymetric survey of the canal (Figure 2) and underwater inspection of approximately 3800 lineal feet of the canal's perimeter. Information collected during the dives is contained within and will be used as input to a structural analysis of the existing bulkhead walls. This information was required since no historical design or drawing information could be located for the existing project. The dive inspection agenda consisted of two individual tasks:

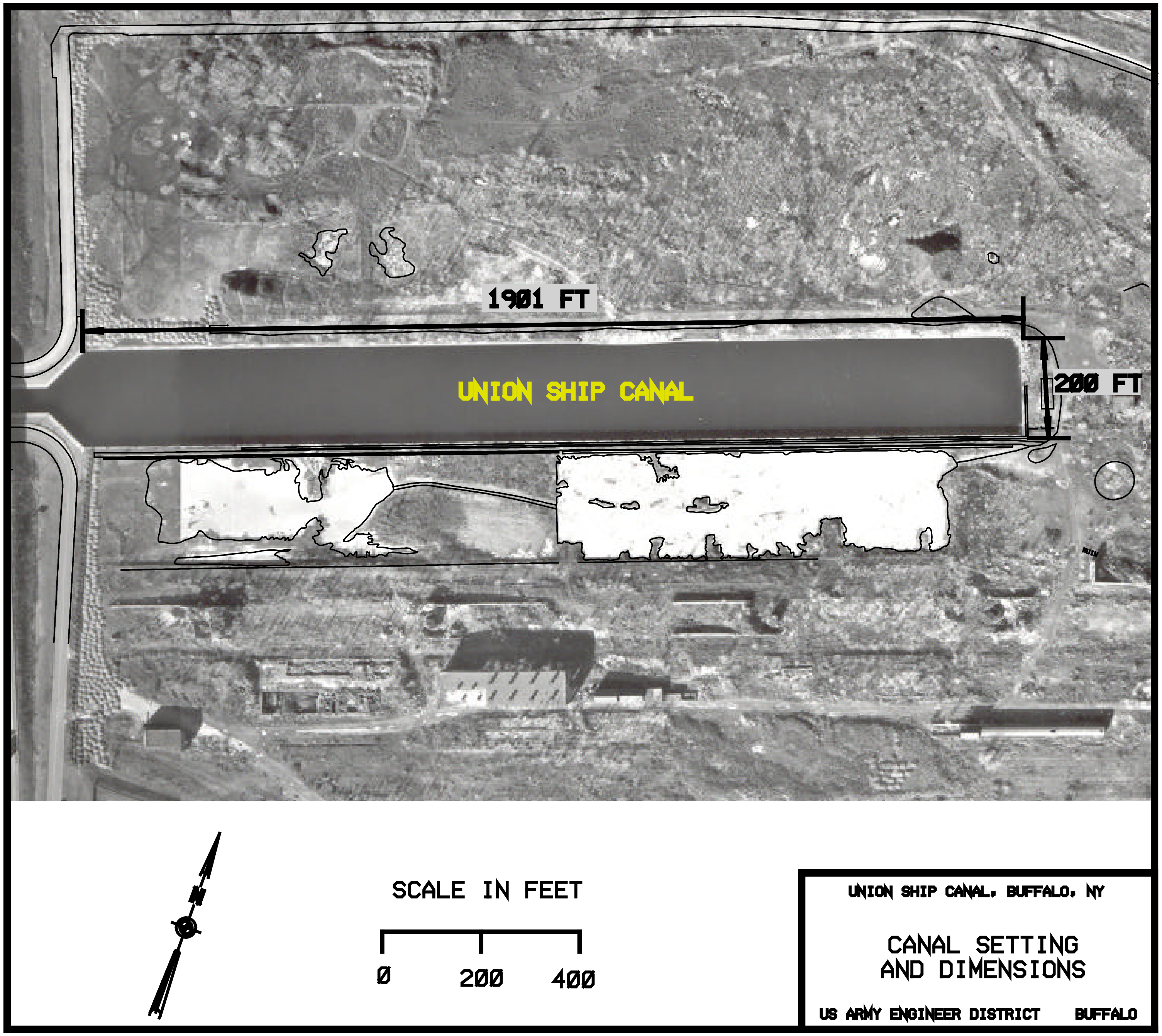
1. Structural inspection dive of the existing canal bulkhead wall.
2. Search pattern dive to map locations of submerged vehicles and large debris.

All work was completed following Corps of Engineers safety and diving regulations.

Structural Dive Inspection

1. An underwater structural dive inspection of the perimeter walls was conducted on 12 September 2001 by the Corps of Engineers Dive Team. Thomas Bender (Diver #1) and Shanon Chader (Diver #2) were the primary divers who performed the underwater inspection. Scott Schlueter was standby diver, James Hasseler was dive supervisor, Dennis Rimer operated the boat and Jim Bruszewski assisted the dive team and collected information on the above water portion of the wall. The underwater inspection was performed using the Corps pontoon boat, surface air supply, and 2-way communication equipment.

a. Site conditions were as follows:
   - Partly sunny
   - Air temperature - 60 – 65 deg.
   - Water temperature - 60 – 65 deg.
   - Underwater visibility - 1 – 10 ft.


1901 FT
200 FT
UNION SHIP CANAL
SCALE IN FEET
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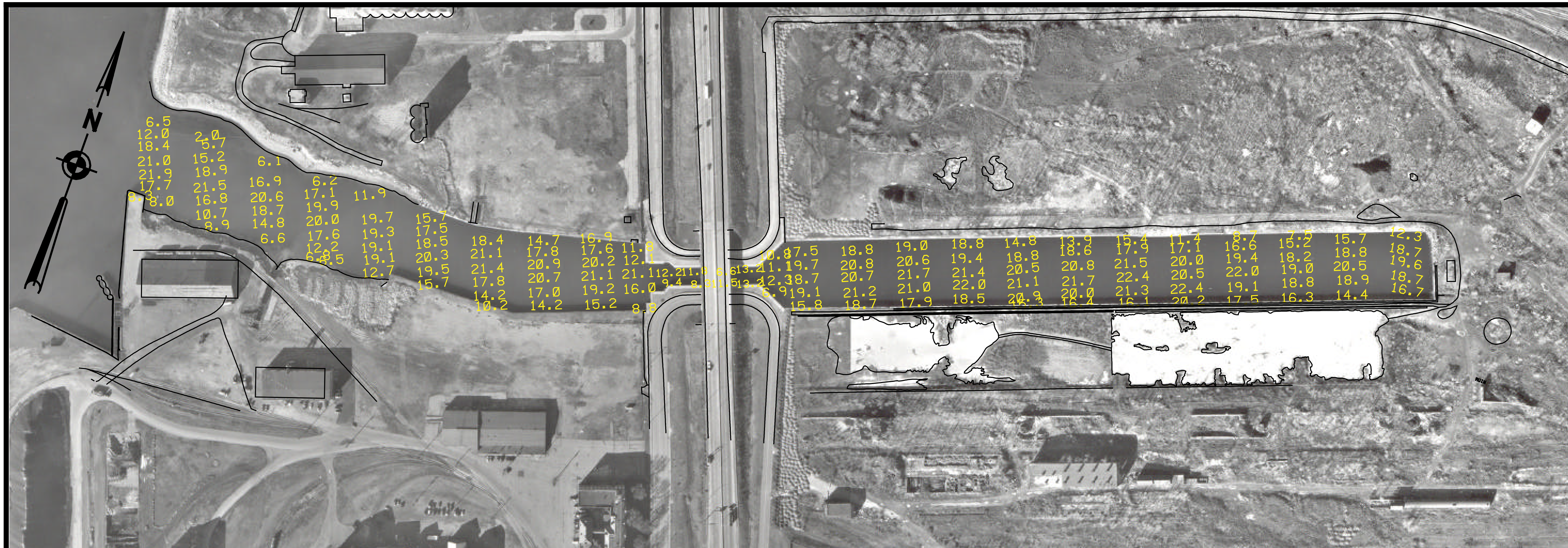
200
400
UNION SHIP CANAL, BUFFALO, NY
CANAL SETTING
AND DIMENSIONS
US ARMY ENGINEER DISTRICT
BUFFALO

FIGURE 1



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12/1/1999 AERIALS

UNION SHIP CANAL, BUFFALO, NY

BATHYMETRIC SURVEY

US ARMY ENGINEER DISTRICT BUFFALO

FIGURE 2

- Turbidity - Varied from low to high
- Bottom Composition - Soft silt, sand, weed beds, and organic matter
- Water depth - Varied between 15 and 22 ft.
- No water current
- No wave action

A compilation of the notes is presented in Tables 1 and 2.

The perimeter dive inspection was initiated at the west end of the north wall (Figure 3) at the location of the terminus of a new steel sheetpile wall (Monolith #1) and ended at the terminus of a new sheetpile wall at the west end of the south wall (Figure 4). The steel sheetpile wall is part of a bridge pier support structure that was placed approximately 10 years ago when the old Father Baker Bridge was replaced with the current low level bridge.

Figure 3. Looking towards the northwest corner of the Union Ship Canal.

Diver #1 descended along the end of the sheet pile to the channel bottom in approximately 20 ft of water. The visible, above water concrete wall extended only to the water surface which at the time of the inspection coincided with the top of an underlying timber crib structure. The timber crib structure was constructed with continuous timbers along its face as opposed to alternating timber and gap construction which was observed along the south wall of the canal. The cribbing exhibited no signs of distress even near the water surface where deterioration is common. Approximately 8 ft off the bottom, the cribbing was founded on a masonry leveling course of stone and mortar. The leveling course varied in thickness (generally 1 to 4 ft.) to accommodate the irregular top surface of the shale bedrock at this location. Below the masonry leveling course, the shale had been excavated several more feet to provide the necessary depth for large draft vessels. The existing vertical shale face was intact and appeared sound as evidenced by the visible drill hole castings. There was generally a small 6 in. to 1 ft. offset between the face of the shale/masonry and the overlying timber crib. A typical section of the north wall at approximate station 8+50 is illustrated in Figure 5. The canal bottom at the wall consisted of a mixture of soft sediments, sand, gravel and cobble size material including concrete spalls that sloped steeply away from the wall over a distance of about 5 to 10 ft. The bottom sediments then leveled out and became primarily soft muddy sediments towards the center of the canal. Sparse patches of vegetation were also observed.

Figure 4. Looking towards the southwest corner of the Union Ship Canal.

Proceeding easterly along the wall similar conditions were observed up to about monolith # 19. Miscellaneous debris was periodically observed on the bottom including a motorcycle, several piles of rock and various trash items. At monolith #19 the shale/masonry was no longer visible partly due to shallower water but mostly due to an easterly dip in the elevation of the top of the shale. Over this distance the top of shale drops approximately 6 ft. Between Monolith #1 and Monolith #19 the cribbing appeared vertical and sound with no discontinuities, voids or damaged areas. However, above water observations indicated that an offset up to 2 ft exists between the face of the concrete wall and the top edge of the timber crib. Since the concrete wall is straight in alignment the cribbing therefore has a very gentle sinuosity which was probably constructed as such. At monolith #20 the crib started to show signs of tipping toward the channel which continued for the next several monoliths (about 200 ft). This created a bowed alignment in the cribbing which also coincided with the above water sections of concrete wall that were missing. The maximum tipping was estimated at about 4 ft from vertical. There was no evidence of the concrete wall sections underwater implying that the sections were removed from above. The next 90 ft of the wall was not bowed nor missing the upper concrete wall but did appear to be tipping slightly (1 to 2 feet from vertical). The timber crib in this area was in overall good condition down to the mud line (Figure 6). The last section of the wall to the northeast corner of the canal (approx. 600 ft) appeared to be vertical and in good overall condition other than the above water concrete was missing (Figure 7). There was no evidence of the failed wall sections underwater again suggesting that the wall had been previously removed . The water depth in this area ranged from 12 to 16 ft and numerous pieces of debris, trash, and miscellaneous steel was observed along with dense vegetation. At approximately 50 ft from the end of the canal the bottom shallowed considerably which terminated the dive.

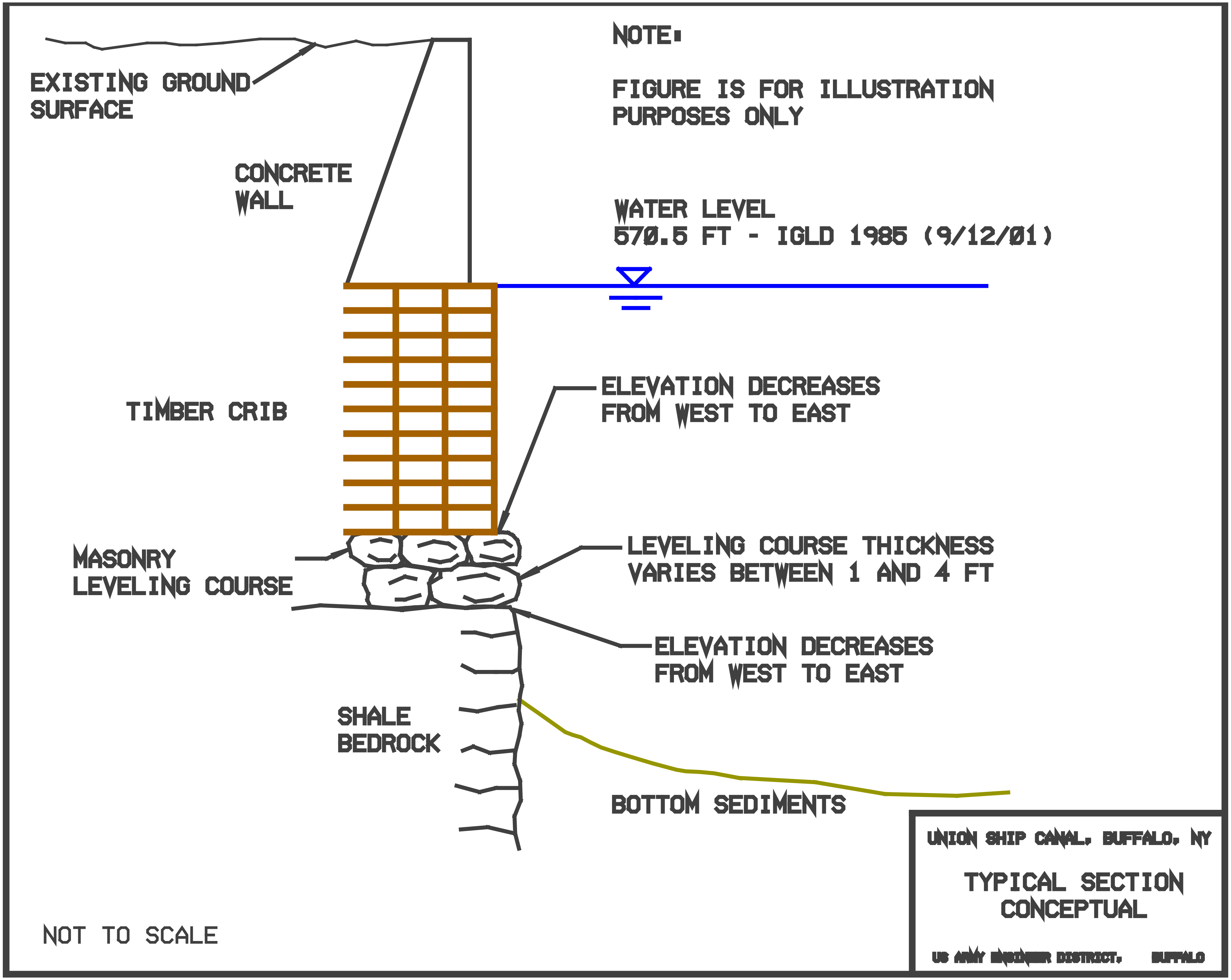


FIGURE . TYPICAL NORTH WALL SECTION NEAR STATION 8+50

Table 1. Union Ship Canal Dive Notes – Structural Assessment North Wall– Surface Air

Monolith	Diver	Date	Notes
1			Masonry – 8 ft above sediment – 20 ft depth
2			Timber crib above masonry capped by concrete
3			
4			
5			Motorcycle located at midpoint of reach
6			1.5 ft offset from concrete cap to timber cribbing
7			
8			Steam roller wheel located
9			
10			Drill hole visible in shale bedrock from excavation
11			
12			
13			
14			Possible masonry leveling course placed in this area – 1 to 4 ft thick
15	Bender	9/12/01	Smooth shale face – highly jointed
16			
17			Crib out 2 ft from concrete
18			Crib out 3 – 4 ft --- 20 ft depth ---Crib 2 ft above mud
19			Crib down to mud line – 18 ft depth
20			Low section – bowed out, crib in good condition – tipping
21			Low section – bowed out, crib in good condition – tipping
22			Low section – bowed out, crib in good condition – tipping
23			Low section – bowed out, crib in good condition – tipping
24			Crib tipping slightly
25			Crib tipping slightly
			16 ft water depth - No concrete this point on – all timber vertical crib
			12 ft water depth at 2/3 the length, remnants of tie rods, evidence of wood cross members 5 ft above the top, top approximately 1.5 ft above the water line – 16 ft water depth Stopped approximately 50 ft from northeast corner due to debris

Figure 6. End of concrete cap, beginning of bowed section.

Figure 7. Continuation of bowed section, end of concrete cap section.

Diver #2 began the second portion of the underwater inspection approximately 50 ft from the southeast corner due to excessive debris that lined the eastern end of the canal. Diver #2 descended and proceeded along the south canal wall in approximately 15 – 20 ft of water. The existing canal bottom along the entire south wall was similar to that of the north wall, it consisted of a mixture of soft sediments, sand, gravel and cobble size material including concrete spalls that sloped steeply away from the wall over a distance of about 5 to 10 ft. The bottom sediments then leveled out and became primarily soft muddy sediments towards the center of the canal. Sparse patches of vegetation were also observed. The existing shale bedrock lining the canal wall varied from 0 ft above the canal bottom at the eastern end to approximately 6 ft above the canal bottom at the western end. This correlates well with observations made along the north wall.

The existing timber crib was constructed using alternating timber and gap design which differed from that on the north wall. The construction of the crib was relatively tight and there was no evidence to suggest loss of fill. Unlike the north wall, the south canal wall was generally sound along its entire length with no evidence of tipping. It had a more uniform face as opposed to the step back design shown in Figure 5. Several areas appeared to have a masonry leveling course that generally followed a similar trend as that of the north wall. The existing canal wall (north and south) appears to be structurally sound below the water line. Diver #2 observed potential minor bowing of the timber crib along the southeast portion of the wall. This should be investigated further before any construction activities take place on or near the wall. Also, undermining of less than 2 ft into the shale base was noted along several sections of the south wall ranging from a couple of feet to 20 feet in length. The undermining is likely due to previous shipping activities in the canal. Since all shipping and industrial activities have ceased in the canal, there should be no danger of increased undermining. Figure 8 illustrates the typical above water condition of the existing southern canal wall. The field notes obtained are presented in Table 2. Since the discernable monoliths ended around # 17, symbols were used as marking points.

Figure 8. Typical condition of the southeast wall above the waterline.

Table 2. Union Ship Canal Dive Notes – Structural Assessment South Wall– Surface Air

Monolith	Diver	Date	Notes
1	Chader	9/12/01	Crib covered with zebra mussels, 18 ft depth
2			
3			
4			Piece of timber crib missing, 5 ft up – discontinuous cribbing.
5			15 ft depth – approx. 8 ft timber bumper on bottom
6			1.5 ft offset of concrete cap over timber – timber sticks out
7			
8			Slight outward bow – 6 inches
9			1 ft offset
10			
11			
12			Intermittent cribbing – good closure and stone fill containment
13			6 inch cylinders, rise to 2 ft off of bottom – 22 ft depth
14			More 6 inch cylinders
15			20 ft water depth
16			
17			20 ft water depth – good fill containment
⮝			Approx. 50 ft to 5 ft wide culvert – 21 ft water depth
			Shale just above culvert, 24 ft bottom of cribbing
			2.5 – 3 ft ledge to bottom of crib
⮟			22 ft water depth
			1 ft undermining for approx. 20 ft
⊙			Sheet piling on bottom
			20 – 25 ft of crib on shale undermined by approx. 6 inches
			Sta. 8+00 water depth is 20 ft, approx. 30 ft undermined by 8 inches
+			Approx. 35 ft of undermining – 1 ft deep – 1 ft of shale above bottom
			Approx. 30 ft from mark – masonry? and shale
			Shale approx. 4 ft from bottom in 21 ft water depth
∝			4 ft masonry wall
			19 ft water depth
			100 ft from end – approx. 6 ft of shale from bottom

Summary

Based on the underwater structural inspection, the majority of the existing south canal wall inspected is structurally stable. However, if any future construction or improvements to the existing wall are slated, further investigation is required. The north wall appears to be structurally stable along the first 1000 ft beginning at the north west corner. The next 200 ft section has no concrete cap and appears to be bowing out and tipping. Before any construction activities or designs are considered for this area, further structural analysis should be completed. The remaining section of canal wall has had the concrete cap removed. Since no documentation exists for the site, it must be assumed that it was removed to decrease the wall loading. Once again, if any construction activities or designs are considered for this area, further structural analysis should be completed.

Recent low lake water levels have exposed a 1 to 2 ft portion of the previously submerged timber crib structure along the entire structure. When submerged, the timber cribbing is protected against wet/dry cycles and dry rot. The exposed timber cribbing above the waterline will continue to deteriorate and lose structural integrity over time. Therefore, the exposed cribbing may require additional protective measures to decrease or prevent deterioration.

Search Pattern Dive

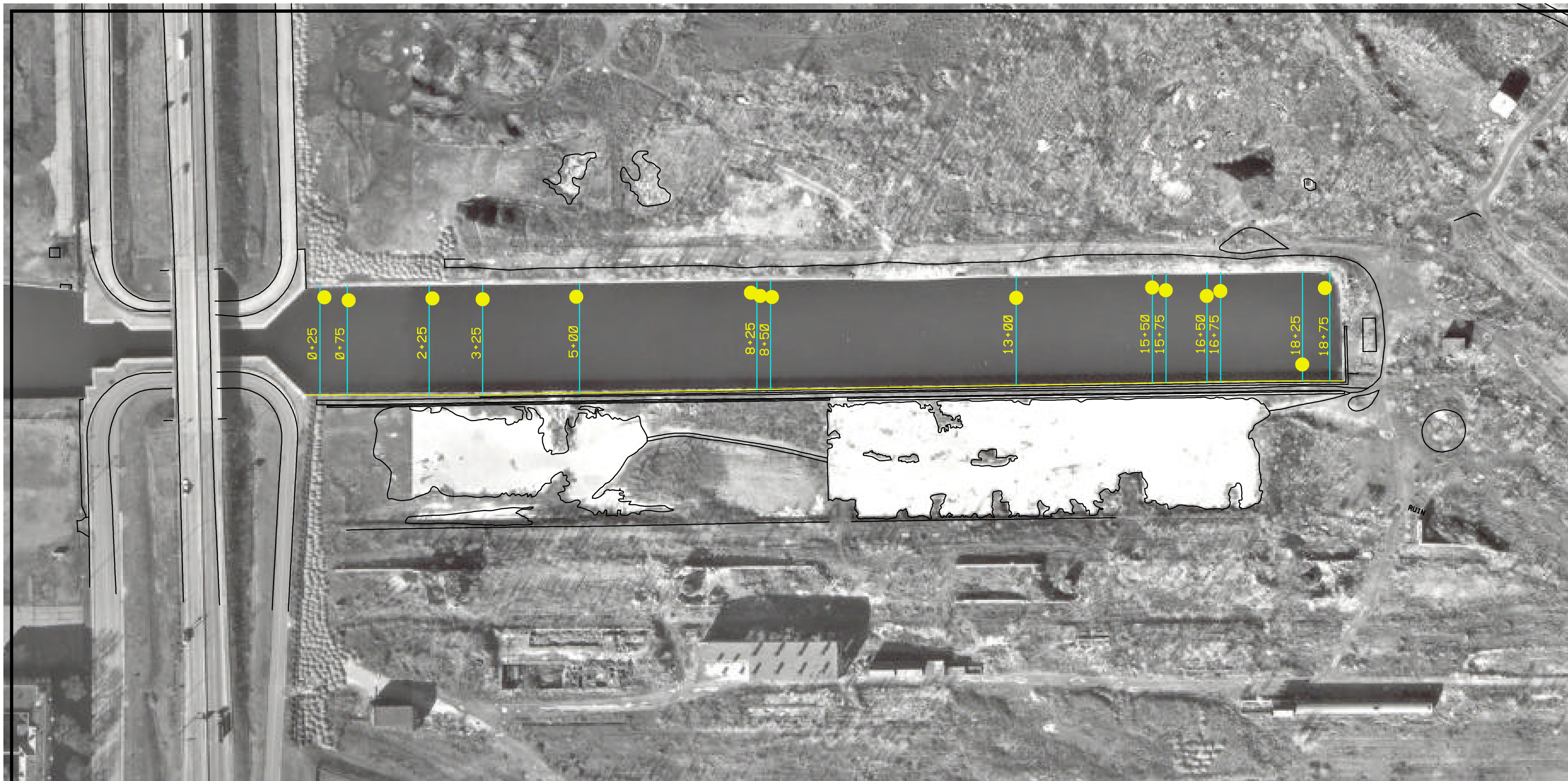
2. The underwater search pattern dive was divided into several dives conducted between 13 September 2001 and 24 October 2001 by the Corps of Engineers Dive Team to locate existing submerged vehicles and other miscellaneous debris. The underwater search was performed using the Corps pontoon boat as a dive platform (Figure 8), SCUBA (Self-Contained Underwater Breathing Apparatus) gear, and a predetermined search pattern.

Figure 9. Corps pontoon boat, crew preparing section lines.

The canal was divided into seventy seven, 25 ft wide (approximate) sections running north/south. A weighted line running north/south across the canal was used to aid in diver navigation. A team of two SCUBA divers used the line as a reference, each scanned a path approximately 12 ft wide over the entire width. The navigation line was moved each time the divers completed a section. After each pass, the divers surfaced and reported bottom conditions, submerged vehicles, and significant debris encountered (Figure 10). Due to the large number of sections, the search pattern dive took place over several days. The notes compiled during these dives are represented in Tables 3 and 4. Figure 11 depicts the approximate location of submerged vehicles encountered during the dive. The following information indicates weather conditions encountered on the given days.

Figure 10. Divers reporting location of submerged vehicles and debris.

a. Site conditions on September 13, 2001:
   - Partly sunny
   - Air temperature - 60 – 65 deg.
   - Water temperature - 60 – 65 deg.
   - Underwater visibility - 1 – 15 ft.
   - Turbidity - Varied from low to high
   - Bottom Composition - Soft silt, sand, weed beds, and organic matter
   - Water depth - Varied between 15 and 22 ft.
   - No water current
   - No wave action



SCALE IN FEET

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12/1/1999 AERIALS

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NOTE:

BASELINE STATIONS AND SUBMERGED CAR LOCATIONS ARE APPROXIMATE

VISIBILITY VARIED DAILY FROM POOR (1 - 3 FT) TO VERY GOOD (10 - 15 FT)

UNION SHIP CANAL, BUFFALO, NY

SUBMERGED CAR LOCATIONS

US ARMY ENGINEER DISTRICT BUFFALO

FIGURE 11

b. Site conditions on October 1, 2001:
   - Partly sunny
   - Air temperature - 60 – 65 deg.
   - Water temperature - 60 – 65 deg.
   - Underwater visibility - 1 – 15 ft.
   - Turbidity - Varied from low to high
   - Bottom Composition - Soft silt, sand, weed beds, and organic matter
   - Water depth - Varied between 15 and 22 ft.
   - No water current
   - No wave action

c. Site conditions on October 10, 2001:
   - Partly sunny
   - Air temperature - 60 – 65 deg.
   - Water temperature - 55 – 60 deg.
   - Underwater visibility - 1 – 15 ft.
   - Turbidity - Varied from low to high
   - Bottom Composition - Soft silt, sand, weed beds, and organic matter
   - Water depth - Varied between 15 and 22 ft.
   - No water current
   - No wave action

d. Site conditions on October 24, 2001:
   - Cloudy/rain
   - Air temperature - 50 – 55 deg.
   - Water temperature - 50 – 55 deg.
   - Underwater visibility - 1 – 15 ft.
   - Turbidity - Varied from low to high
   - Bottom Composition - Soft silt, sand, and organic matter
   - Water depth - Varied between 15 and 22 ft.
   - No water current
   - No wave action

Summary

The search dives were completed over a period of several days with varying site conditions as indicated in the above text. The search dives facilitated location of 15 submerged vehicles, several vehicle parts, and various other miscellaneous timber, metal, and debris. All submerged vehicles and miscellaneous items located during the dives were recorded and are listed in Tables 3 and 4.

Table 3. Union Ship Canal Dive Notes – Search Pattern

Divers	Date	Station	Notes
Bender/Chader	9/13/01	0+00	Few boulders, large number of weeds, no significant objects
		0+25	Vehicle 10 ft from NORTH wall, Heavy clumps of weeds, misc. boulders, 6 ft long PVC pipe
		0+50	Vehicle parts (front quarter panel, wheel) 15' from NORTH wall, 6 ft diameter at wall
		0+75	Vehicle – 25 ft off NORTH wall, 4-door all glass inside (light color, been in water a couple of years)
		1+00	Nothing, few weeds, low visibility
		1+25	Bicycle near NORTH wall
		1+50	Silty bottom, few weeds, nothing found
		1+75	Nothing
		2+00	Structural Steel @ angle 5 ft out of mud @ SOUTH end, large
		2+25	15 ft east of line, Red Ford Mustang 2-door, standard transmission
Rimer/Schlueter	9/13/01	2+50	Moped against E. wall, washer halfway on its side, bench seat, poor visibility
		2+75	Nothing
		3+00	Clear bottom, bicycle & home safe @ NORTH wall
		3+25	Blue minivan
		3+50	Clean pass, nothing found, poor visibility in center
Chader/Schlueter	10/1/01	3+75	Nothing, no weeds, piece of I-beam near SOUTH wall
		4+00	Nothing but timbers 30 ft off edges
		4+25	Tires near NORTH wall, small patches of weeds
		4+50	Bedrock halfway (5-6 rocks, 3-5 ft wide)
		4+75	Rocks, 60 ft out from NORTH wall air cylinder, 40 ft off SOUTH wall railroad vehicle axle w/ wheels, slab of cut stone
		5+00	Tire and vehicle axle, more bedrock
		5+25	Timbers (4 ft long)
		5+50	Random bedrock
		5+75	Three tires near SOUTH wall, steel pipe 3 – 5 ft long, steel plates
		6+00	Vehicle tires near NORTH wall, tires 50 ft from SOUTH wall, scrap steel
		6+25	Scrap steel
		6+50	Weeds, bumpers from wall, old scrap metal 20 ft from SOUTH wall, large size timber
		6+75	25 ft off SOUTH wall, 2" x 6' steel I-beams – a lot smaller by south wall
		7+00	Scrap metal 20-30 ft from SOUTH wall
		7+25	I-beam scrap, NORTH side much cleaner
Bender/Rimer	10/1/01	7+50	Soft muddy bottom, sparse vegetation
		7+75	Tire 5 ft from SOUTH wall
		8+00	Tires (bumper tires near SOUTH wall & another 1/3 way out)
		8+25	Chevy Astro Van 50 ft off NORTH wall (upside down), another undistinguishable vehicle upside down
		8+50	Camaro, 2 door, both open, sitting upright with a spoiler on back, right near NORTH wall
		8+75	Nothing, scrap metal
		9+00	Nothing, few weeds
		9+25	Tire near SOUTH wall, mud sediments

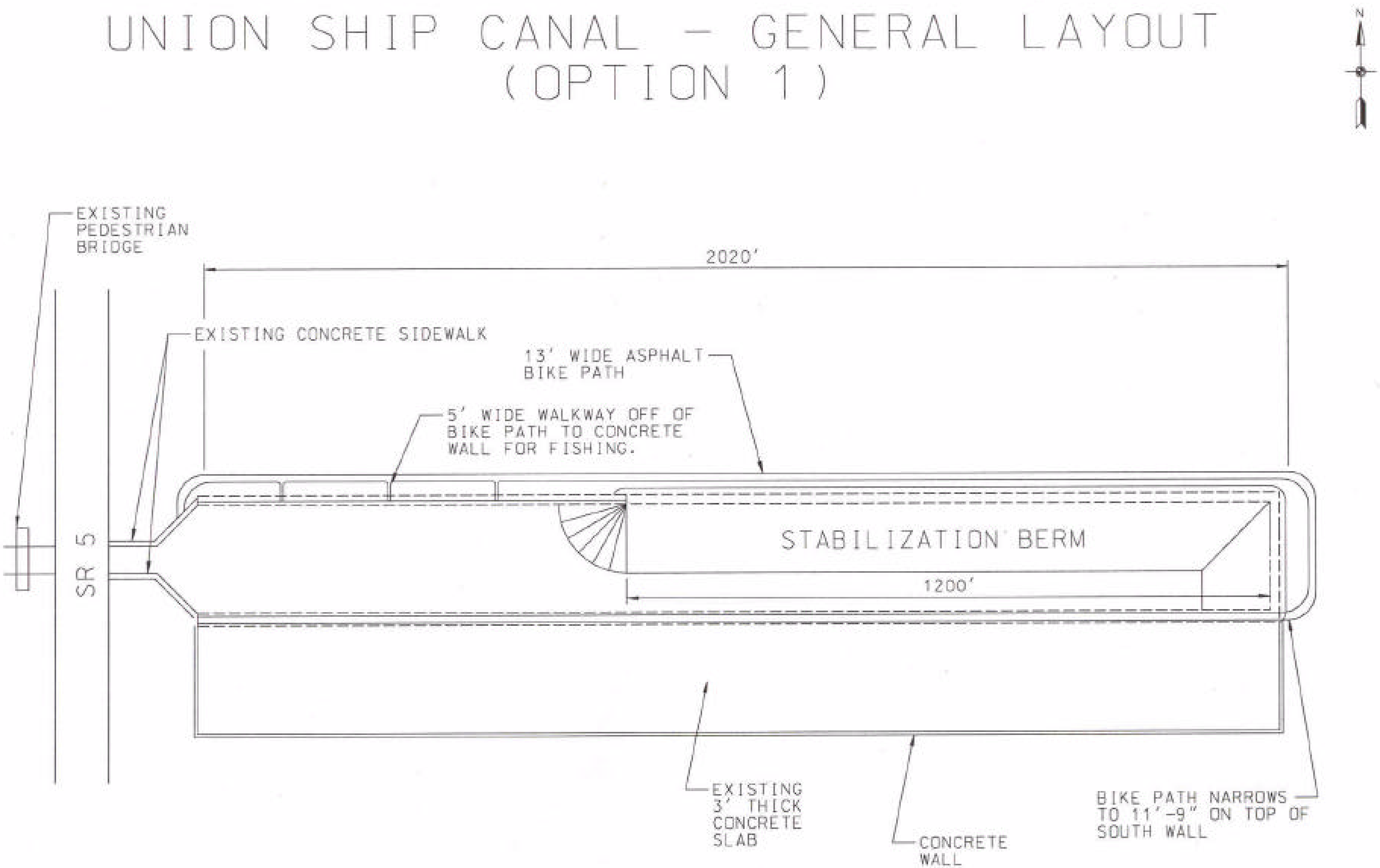
Note: Stationing is approximate

Table 4. Union Ship Canal Dive Notes – Search Pattern

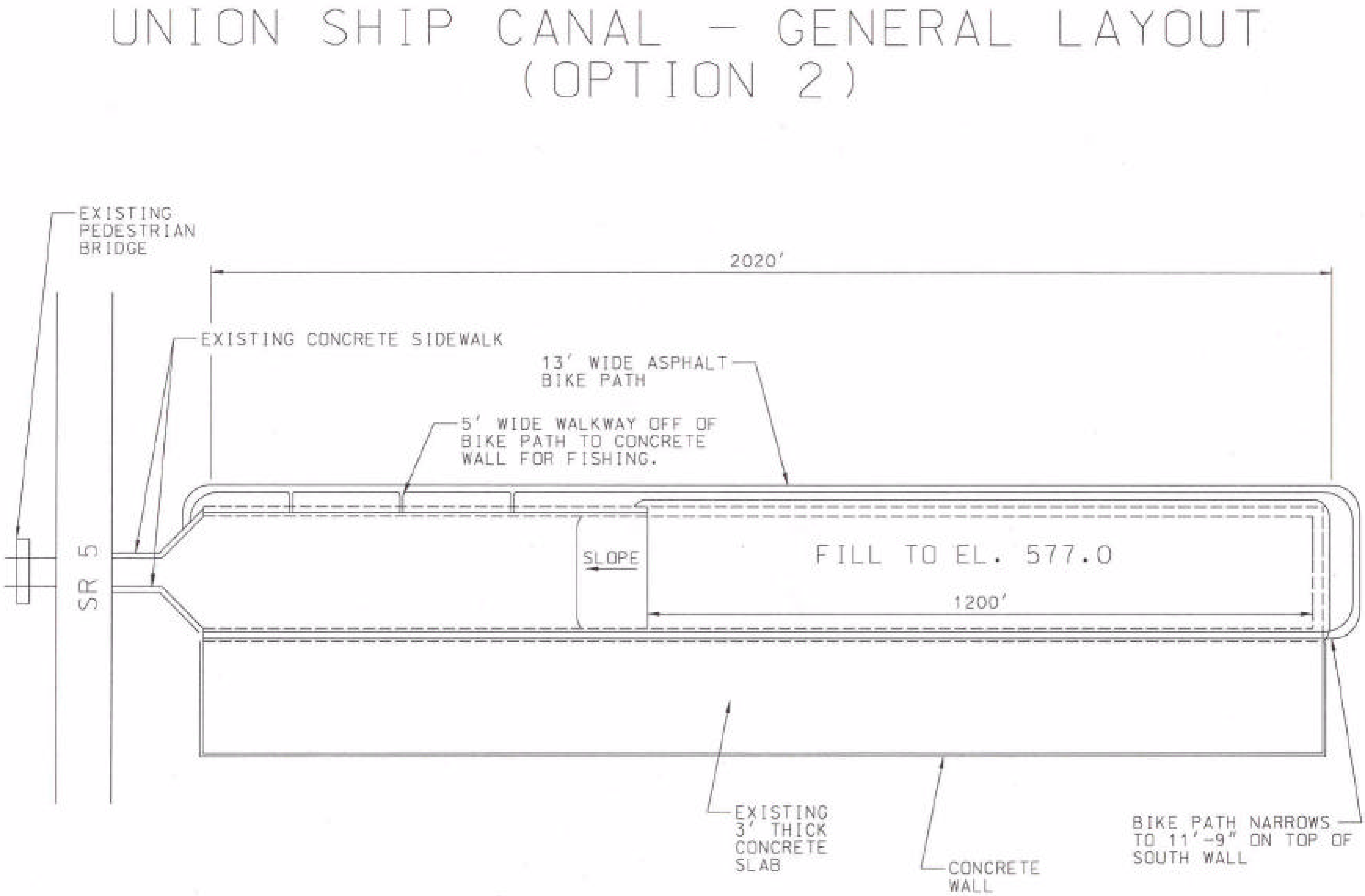
Divers	Date	Station	Notes
Bender/Rimer	10/1/01	9+55	Angle iron near SOUTH wall, rocks
		9+75	Piece of railroad track, no weeds
		10+00	2 tires near SOUTH wall
		10+25	Large weed beds, angle iron (10 ft long) almost vertical against NORTH wall, 1 large tire near NORTH wall
		10+50	Tire near SOUTH wall, small tire in middle of canal
		10+75	Nothing, soft mud bottom, weed beds
		11+00	Small tires
Chader/Bender	10/10/01	11+25	Vehicle tires near SOUTH wall, a few wooden palettes
		11+50	Small tire in middle of canal, nothing else
		11+75	20 ft long buried timber 1/3 way from SOUTH wall, tires, and boat cushion near NORTH wall
		12+00	Nothing, small tire
		12+25	Tires near NORTH wall, concrete debris near NORTH wall, boat anchor in middle of canal
		12+50	Nothing
		12+75	Few bottles of Black Dog Ale
		13+00	Vehicle, plates retrieved, 50 ft from NORTH wall, 20 ft long railroad rail, steering column from vehicle
		13+25	Tire near NORTH wall
		13+50	Three timber bumpers 50 ft off NORTH wall
		13+75	Nothing
		14+00	Nothing
		14+25	Nothing
		14+50	Tire off SOUTH wall
		14+75	Denser weed growth near NORTH wall
		15+00	Nothing
		15+25	Timbers near SOUTH wall
		15+50	Small minivan 25 ft off NORTH wall
Chader/Rimer	10/24/01	15+75	Vehicle located approx. 30 ft from north wall
		16+00	Nothing
		16+25	Nothing
		16+50	Vehicle – approx. 40 ft from North Wall
		16+75	Vehicle (old) located approx. 20 – 30 ft from north wall – upside down
		17+00	Small timber and tree debris
		17+25	Nothing
		17+50	Nothing
		17+75	Large tree approx. 20 ft from north wall
		18+00	Nothing
		18+25	Weeds near wall – misc. timber and debris
		18+50	Vehicle – Firebird located approx. 30 ft from South Wall
		18+75	Thick weeds, lots of trash
		18+75	Minivan located approx. 30 ft from north wall – lots of misc. debris
		19+00	Too shallow to swim – lots of misc. debris at east end of canal
		License plates recovered from vehicles	
		7RS 485 - 1996 Ford Winstar	
		U83 5X5 - 1995 Jeep Cherokee	

Note: Stationing is approximate

UNION SHIP CANAL - GENERAL LAYOUT (OPTION 1)



UNION SHIP CANAL - GENERAL LAYOUT (OPTION 2)



US Army Corps of Engineers
Buffalo District

Union Ship Canal
Buffalo, New York

Structural Analysis

Appendix B

November 2002

1. AUTHORITY

This report was prepared under Section 22, of the Water Resources Development Act (WRDA) 1974, as amended, which allows the Corps of Engineers to provide technical assistance to support state preparation of comprehensive water and related land resources development plans, including watershed and ecosystem planning. Section 22 is also known as Planning Assistance to States (PAS). Assistance is given on the basis of state requests and availability of Corps expertise rather than Congressional study authorization procedures. Cost sharing is based on a 50% Federal and 50% non-Federal basis.

The investigation to determine the Structural integrity of the Union Ship Canal, Buffalo, NY was undertaken in response to a request from Erie County and the City of Buffalo. All work performed under the Section 22 Authority, was identified in an Agreement which was executed between the Corps of Engineers and the City of Buffalo on 15 July 2001.

2. STUDY PURPOSE AND SCOPE

The purpose of utilizing the Section 22 Program was to determine the structural integrity of the structures supporting the fill around the Union Ship Canal. The background investigations which were performed in support of this work included the following: Dive Inspection of the Canal Walls, Drilling and Boring Program, and a Geotechnical Analysis. These investigations were required to make recommendations on the structural integrity of the walls. In addition the above mentioned studies, Sediment Analyses and a Dive Inspection of the Canal bottom were also completed.

3. LOCATION

The Union Ship Canal is located at the southern limit in the city of Buffalo, Erie County, New York. The Union Ship Canal was constructed in 1905 and was originally named the Goodyear Slip. The Union Ship Canal was then used by Hanna Furnace until operations ceased at the site in 1982. Current dimensions include an approximate length of 2,000 feet and width of 200 feet. The location map is included as Figure 1.

4. EXISTING CONDITIONS & PROJECT DESCRIPTION

"The Union Ship Canal, close to the city line at the Route 5 bridge, began its existence as the Goodyear Slip, begun in 1903. The slip was built to service Frank and Charles Goodyear's new iron plant, the Buffalo & Susquehanna Iron and Coal Co.

The Goodyear Slip was completed in 1905 to a length of 4,000 feet. Its wharves serviced the plant and provided for the unloading and shipping of iron ore and limestone to other points on the Goodyears' Buffalo & Susquehanna Railway.

…Eventually the Iron plant itself ended up in the hands of Hanna Furnace, which produced pig iron at the site until 1982. The entire plant was scrapped over the next several years."[1]

Both sides of the canal were used for unloading freighters to stockpiles or railroad cars. Brown electric unloaders had 5-ton capacity clamshell buckets.[2]

Currently, the remaining portion of what is now known as the Union Ship Canal is approximately 2000 feet long by 200 feet wide. An initial site inspection and separate dive inspection was conducted in 2001, along with test pits and borings to characterize the structures at the site. From the above inspections, the structure appears to be a pinned timber crib structure with concrete cap. The design of the concrete cap and width of the timber crib structures seems to vary. Portions of the cap are anchored with horizontal tie-rods to an unknown anchorage system. The timber crib extends from top of rock to approximately 1' below water on the date of the dive inspection or approximately El. 569.4 (IGLD 1985).

The timber crib observed in the dive inspection appears to be in good condition around the entire canal. Portions of the North Wall, however, show evidence of horizontal and/or rotational movement of the timber cribbing. Additionally, some locations are missing the concrete caps (Reach F and the portion between Reach G & I), presumably due to failure. It is likely that these caps were intentionally removed as no evidence of them was found during the dive inspection. The South Wall shows no evidence of misalignment or movement. With the exception of the concrete cap being missing from one half the length of the East End Wall, the remaining portion shows no sign of misalignment. The cribbing was not observable during the dive inspection as the diver could not safely approach the wall due to debris.

[1] Buffalo's Waterfront, a Guidebook. p. 52-53, Ed. Timothy Tielman, 1990.
[2] Images of America, Buffalo's Waterfront. p. 47, T. Leary & E. Sholes, 1997.

5. Structural Analysis

Due to the lack of drawings available, all dimensions for the structures were measured in the field from the dive inspection and test pit investigations or assumed based on information collected. No information is available about the design of the timber cribbing or the species of the wood. Portions that were visible appeared to be pinned construction which is typical for timber cribbing of that time period. With the exception of the portions above water, the dive inspection observed the cribbing to be tight and in good condition. Wood that is not subjected to alternating wetting and drying will not deteriorate. The timber crib structures have been continuously submerged and show no sign of structural deficiency and are therefore assumed to be acceptable for continued service. As the condition of the cribbing appears good, and as the complexity of an analysis of the timber cribbing combined with the lack of information would make any results of such an analysis highly subjective, the structure was analyzed to behave as a rigid monolith. The analyses were therefore, limited to stability analyses.

Soil properties were determined from borings taken for the geotechnical investigation. Subsurface profiles were determined based on the elevations of the interface between the granular fill and clay glacial deposits at various distances from the wall and the assumed construction method. Due to the nature of the shale found at the site, no samples were obtained large enough to perform compression testing on. The portions of the shale that were observable during the dive inspection did not show evidence of failure. Due to the lack of samples from which to determine unconfined bearing strength testing, that the rock that was observable appeared to be in good condition, and due to the fact that the intended use would not increase the loading beyond what the wall has already seen, bearing was not checked as a failure mode. The stability analyses were limited to overturning and sliding. The concrete caps were analyzed separately and in conjunction with the timber cribbing for the structure as a whole.

The calculated factor of safety for sliding and percentage of base in compression were compared to both current design requirements for new structures and those for existing structures. Where the structure met the requirements for the design of new structures or the structure met the requirements for existing structures and showed no evidence of misalignment or failure, the stability was considered acceptable. Where the structure neither met current or existing structure stability requirements or where it showed evidence of misalignment, the structure was considered to require rehabilitation.

6. Rehabilitation Alternatives

Several options were initially considered for structures that required stabilization. These alternatives include cantilever sheet pile walls and sheet pile walls with horizontal tie-rods to concrete deadmen or to a sheet pile anchor wall; replacement of the concrete cap where missing and tied back with similar systems or soil anchors; replacement of the missing portions of the concrete cap; and stabilization with a berm of either stone or dredged material.

Due to the contaminated nature of the soil and sediments surrounding the canal, it would be preferable to minimize disturbance to the soil. A cantilever sheet pile wall would have the benefit of not requiring anchorage and would therefore not require any excavation into the soils surrounding the canal or disturbance of sediment within the canal. Based on the likely minimal depth of sediment, and due to the weak nature of the silt, it was determined that there would be insufficient material to support a cantilever sheet pile wall. An anchorage system would therefore be required for a sheet pile wall design. Any option that required installation of a deadman or sheet pile anchor wall system, would have significant amounts of excavation and associated costs of disposal of the contaminated material. Should soil anchors be used, this excavation would not be necessary and the disposal of soil would be limited to that in the drill holes. The toes of the sheet piles would have to be pinned into the bedrock due to insufficient sediment as noted above. Due to the lack of data obtained on rock strength, it is not known if pinning is a viable option. A sheet pile wall however, would be out of character for the site and would detract from its aesthetics. Due to the significant length of the wall to be replaced, the cost would be expensive. Additionally, depending on the anchorage system selected, this could require significant amounts of excavation, which as noted above, is undesirable.

Replacement of the missing concrete caps was only briefly considered. Significant amounts of excavation would be required to access these locations for construction. Even if the cap itself was anchored back, this would not address the stability concerns of the structure as a whole. The new concrete would be difficult to match in color, texture and aggregates, and would therefore have a negative aesthetic impact.

Stabilization berms were therefore chosen as the recommended alternative. The berms could be constructed of dredged material or dumped stone or other environmentally suitable fill material. The allowable slopes for the berms and therefore the required quantities, would be dependent on the material selected. If dredged material was used, the concrete caps of the portions of wall necessitating stabilization would be removed and the above water portion of the backfill, sloped to a natural stable angle. If a stone berm is used, the concrete structures can remain in place. The slopes will be protected with rip-rap between –3 feet and +8 feet LWD for changes in water level. The berms will have the benefit of being a simple to install repair that will not disturb existing soil in front of or behind the existing structures (with the exception of removing the concrete caps and associated backfill if dredged material is used). If stone berms are chosen, this will have the benefit of providing improved fish habitat which will benefit the recreational fishermen. It was requested to evaluate the possibility of having a berm across the width of canal and filling in behind this berm. The fill material would have to be brought up to an elevation of 577.0 to prevent frequent overtopping of the fill. This elevation corresponds to the average annual maximum elevation for the years between 1994 and 2002 that were available from the National Oceanographic and Atmospheric Association. This elevation would allow the material to be seeded and the area used for recreation. The quantities of material however would be much greater than providing a berm around the perimeter of the canal where required. The riprap slopes above water, would be aesthetically pleasing. Figures depicting the proposed repairs appear later in this appendix.

The condition of the concrete is, in general, good. Some cracking and spalling is evident on the top waterside corners of the structures. The tested strength of the concrete was a minimum of 6000 psi. which is much higher than usual new construction designs. To be appropriate for public access, handrails will need to be provided on the top of existing concrete caps to remain. Handrail will be required on both sides of the concrete cap for Reach C, due to the five foot drop to the top of the concrete slab. It is recommended that the handrails be set back approximately 2' from the edge of concrete and around recesses for the bollards. This would have three benefits. First, the additional standoff distance would provide additional safety to the public. Secondly, this would eliminate the need to repair the concrete along the structure corners as the anchorages for the handrails would not be in the cracked regions. Concrete repair would involve chipping out cracked concrete and repairing with new concrete which would have to be anchored to the existing structure to keep it from spalling off again. This could be a significant cost. The third benefit would be to aesthetics. As noted previously, concrete repairs would be difficult to match to the concrete of the existing structure and would therefore be an eyesore. Additionally, leaving the bollards in place maintains the historic character of the canal.

General layout sketches are provided depicting the recommended stabilization berm options.

7. Pertinent Data

Essential data on cost, physical features, project purpose, and controlling elevations.

- An estimate based on the specific proposed design is being currently conducted and will be included in this report when complete.

- Based on observed elevation of the water during the dive inspection and the recorded USGS average water level for that date, the top of the concrete caps appear to be at el. 579.4 (+10.2 LWD - IGLD 1985). The tops of the timber crib below the concrete caps are 10' below the top of the concrete cap or el 569.4 (+0.2 LWD - IGLD 1985). The cribs are founded on shale that tends to drop in elevation from 24 feet (el. 555.4) to 35 feet (el. 544.4) below the surface from West to East.

- Soil and rock properties were determined by testing of samples obtained from the geotechnical investigation, attached.

8. References

1. "Site Investigation Report, Union Ship Canal Rehabilitation Project, Buffalo, New York", SJB Services, Inc., April 2002 (SJB-BD 02-016)

2. "Union Ship Canal, Geotechnical Report for Structural Analysis of Canal Walls", U.S. Army Corps of Engineers, Coastal/Geotech Team, 17 June, 2002.

3. "Retaining and Flood Walls", EM 1110-2-2502, 29 September, 1989.

4. "Design Manual for Segmental Retaining Walls" National Concrete Masonry Association, 1993.

5. "Basic Soil Mechanics" R. Whitlow, 1990.

6. "Stability Criteria for Existing Concrete Navigation Structures on Rock Foundations", ETL 1110-2-310, 17 December, 1987.

9. Engineering Studies, Investigations and Design

a) Preliminary Investigation:

An initial site inspection was performed to begin characterizing the site and structures. A general description of the structures that was visible was made and possible construction methods theorized for the portions not visible below ground. A later dive inspection was performed to confirm or refine the assumptions made from the initial site inspection concerning the construction and condition of the structures below water, and to determine what analyses would be required. Finally, due to lack of information on the structure dimensions, several test pits were dug to reveal buried portions of the structures. Results of these investigations are attached.

b) Analysis

After determining that portions of the canal wall were in need of rehabilitation, the team began to consider possible repair or replacement alternatives to portions of the canal as described above. Each section of the canal was analyzed and repairs designed according to appropriate engineering methods, manuals, regulations, and standards.

1) General

The wall was broken into six separate reaches for the analysis and design based on observations from the preliminary site visit and geotechnical investigation. The reaches were separated because of varying geometric configurations and soil parameters. For simplicity, only one load case was used for the analysis. This load case was the existing condition at each section with the appropriate soil loads applied, and a uniform surcharge representing snow cover of 100 psf. Due to the frequency of and long times that snow cover can remain, this load case was conservatively considered a "Usual" load case for determination of the required stability parameters. As the future intended use by the City does not indicate significant additional loads, no other externally applied load cases were considered. Brief consideration was given to including a seismic load case, however it was decided that this would not provide beneficial data. Due to the pinned nature of the timber cribbing, the structure would be highly flexible. The response to a seismic event would be difficult to predict and highly variable. The complexity of the analysis would be far beyond the scope of the analyses intended for these structures. This region is in a relatively low seismic zone so seismic forces would likely be small. Due to the flexibility of the structure, a failure might result in large deflections of the structure, however catastrophic collapse would be unlikely. Even if this should occur, the 200' buffer zone provided for recreation would be more than sufficient to prevent the failure of the canal wall from contributing to the damage sustained by structures outside of this zone due to the same seismic event.

Data recovered from the soil borings would indicate that the construction for the canal involved excavation down to rock where the timber cribbing was either built directly or potentially floated in at a later date. The excavation would have been through the clay glacial deposits. The excavated regions sloped away from the base of the structure and were backfilled with coarse grained fill. As no significant additional surcharge loads are anticipated from the intended use of the area surrounding the canal, it was determined that use of drained soil parameters would be appropriate for the clay material. Stability analyses for the structures included stability for the concrete cap individually, and for the combined concrete cap and timber crib structure. For the former, soil parameters for the coarse grained fill were used in the analyses. For the latter case, a simplifying assumption was employed of a single soil layer. The parameters for this soil layer were derived from the more conservative value for each parameter from the coarse grained and clay soil deposits; i.e. the highest moist or saturated unit weights, lowest angle of internal friction. Lateral loads from the soil parameters for the analyses were determined using Coulomb's lateral earth pressure coefficient method, modifying the parameters with a Strength Mobilization Factor to approximate at rest earth pressures as appropriate. These results were used to perform simple overturning, and sliding stability calculations.

For walls that were considered to be unstable, an analysis was performed assuming that a stone berm is placed on the canal side of the wall for stabilization and passive resistance considered. The berm is assumed to extend up to the top of the timber crib structure, and the slope of the berm adjusted until the structure meets the stability requirements for existing structures. A general analysis was later performed and applicable to all reaches assuming that clean dredged fill material was used to create the berm rather than stone. This berm was assumed to be installed at an angle that would provide a stable slope, and continued up out of the water with the concrete cap being removed. The stone berm could achieve, in most cases, stability of the cribbing up to existing structure standards, at a steeper slope than the natural angle of the dredged fill material. Therefore more dredged fill material would be required than for the stone berm. Material and delivery costs would weigh highly in the selection between the two alternatives.

10. Analysis Results

Results of the analyses corresponded well with observations in the field of portions of the wall that appeared to be in distress. Namely these were the portions of the canal where the concrete cap was missing (Reach F and the portion between Reaches G & I), and where displacement of the crib was noted (Reach G and the portion between Reaches G & I). Results are provided below in tabular form. Reaches C and H should be adequate for the intended purpose and loadings for passive recreation. Reaches D, F, G & I will need a berm placed in front of the structures for stabilization. Without stabilization, progressive failure of these structures is likely to continue.

1 Reach	Existing Stability Results				Design Berm		RepairedStability Results		
	Top Cap		Crib		Soil	Stone	Crib		
	F.S. Sliding	Base Comp.	F.S. Sliding	Base Comp.	Slope	Slope	F.S. Sliding	Base Comp.	
C	7.34	100	1.33	79.4	------	------	------	------	2
D	2.71	100	0.9	34.5	9	15	2	75.7	
F	------	------	0.71	13.7	9	9	1.9	72.8	3
G	1.53	100	0.98	54	9	9	1.9	75.5	4
Btwn G & I	Assumed similar to Reach I				9	9			
I	2.29	100	1.27	62.5	9	9	2.03	73.7	5
H	2.29	100	1.41	100	------	------	------		6

Notes:

1. Letter designation for reaches corresponds to test pits from Geotechnical Report. Not all test pit locations resulted in design reaches.

2. F.S. for sliding of crib is not met for "normal" load case, however it is met for an "unusual" load case (1.33). Assumptions made for analyses are conservative, i.e. analysis is for tallest portion of wall, passive soil is neglected, most conservative va

3. Percent base incompression is not met for crib with berm. Assumptions made for analyses are conservative, i.e. most conservative value of soil parameters from each soil layer selected, snow load considered "normal" load case. Placement of the berm and c

4. Tie-Rods were considered effective for overturning stability of the top cap and middle section. Load in tie-rods is only approximately 4% of the conservatively assumed capacity of the tie-rods. Capacity of the tie-rods is insufficient to provide stabili

5. Percent base incompression is not met for crib with berm. Assumptions made for analyses are conservative, i.e. most conservative value of soil parameters from each soil layer selected, snow load considered "normal" load case. Tie-rods were not found in

6. Tie-rods were considered effective for crib overturning and sliding analyses. The required load in the tie-rods used for sliding analyses was limited to the allowable stress in the steel. The allowable stress however, is only 60% of the ultimate strengt

Union Ship Canal, Buffalo, NY

US Army Corps
of Engineers®
Buffalo District

Union Ship Canal
Buffalo, New York

Stability Analysis

Appendix C

July 2002

General Analysis Methodology

The stability analyses herein include overturning and sliding calculations. Due to the nature of the shale foundation, sufficiently sized samples were not obtained to allow testing. As the future use loads are not anticipated to change from the present state, and as the rock layers that were visible during the dive inspection appeared to be intact, it will be assumed that the rock has sufficient capacity. As the cribs are founded on the shale, it is unlikely that a global slope stability failure will occur with a failure plane through the rock, and therefore these analyses will not be considered.

The stability of the structure was analyzed both by its individual components and as a whole. There is no known structural connection between the cap and the timber cribbing. The concrete was likely cast in place and therefore, there would be some interlocking as the concrete filled voids near the top of the crib fill and cured, however these effects would be impossible to quantify and have been ignored. The concrete cap is first assumed to act as its own gravity structure and checked for overturning and sliding at the cap/crib interface. As the condition of tie-rods and anchorages can not be known, if tie-rods are known to exist in a reach, they are ignored for this portion of the analysis unless the structure is not stable without their use.

The stability of the entire cap/crib combined structure is checked for overturning and sliding at the crib/rock interface. If the structure does not meet stability criteria, it is re-checked assuming that the anchors are effective (if existing). The analysis checks the capacity of the anchorages based on an assumed strength for the anchors. The internal stability of the structure, i.e. sliding of the crib below the restrained upper cap is analyzed per the recommendations of chapter 5.7 of the Design Manual for Segmental Retaining Walls.

Considered options for stabilization of the structures included sheet piling and/or horizontal tie-rods to some type of anchor wall. Due to the highly contaminated nature of the material surrounding the canal, it would be undesirable to do any excavation that was not necessary. The use of sheet piling would be complicated by the lack of sufficient overburden to provide fixity for the toes of the sheets which would therefore need to be pinned. Sheet piling would have to extend far enough above the waterline so that the anchorage system did not penetrate the timber piling as the wood timbers and loose fill would be difficult to drill through. The sheet piling above water would not present an aesthetically pleasing view. It was determined therefore that the best method for stabilization would be a berm in front of the structures. Calculations were performed to determine the required slope of the berm to provide adequate stability. The top portion of the berm was assumed to be stone so that a higher slope could be achieved. A calculation was also performed assuming that the berm was simply dredged material and placed to a naturally stable slope with the concrete portions of the structures removed above the waterline. In some instances, the slopes of the rock berm are close to that calculated for the natural slope so there would be little benefit in quantities to use rock over dredged fill. The cost savings of using either method would have to be evaluated.

References:

1. "Site Investigation Report, Union Ship Canal Rehabilitation Project, Buffalo, New York", SJB Services, Inc., April 2002 (SJB-BD 02-016)

2. "Union Ship Canal, Geotechnical Report for Structural Analysis of Canal Walls", U.S. Army Corps of Engineers, Coastal/Geotech Team, 17 June, 2002.

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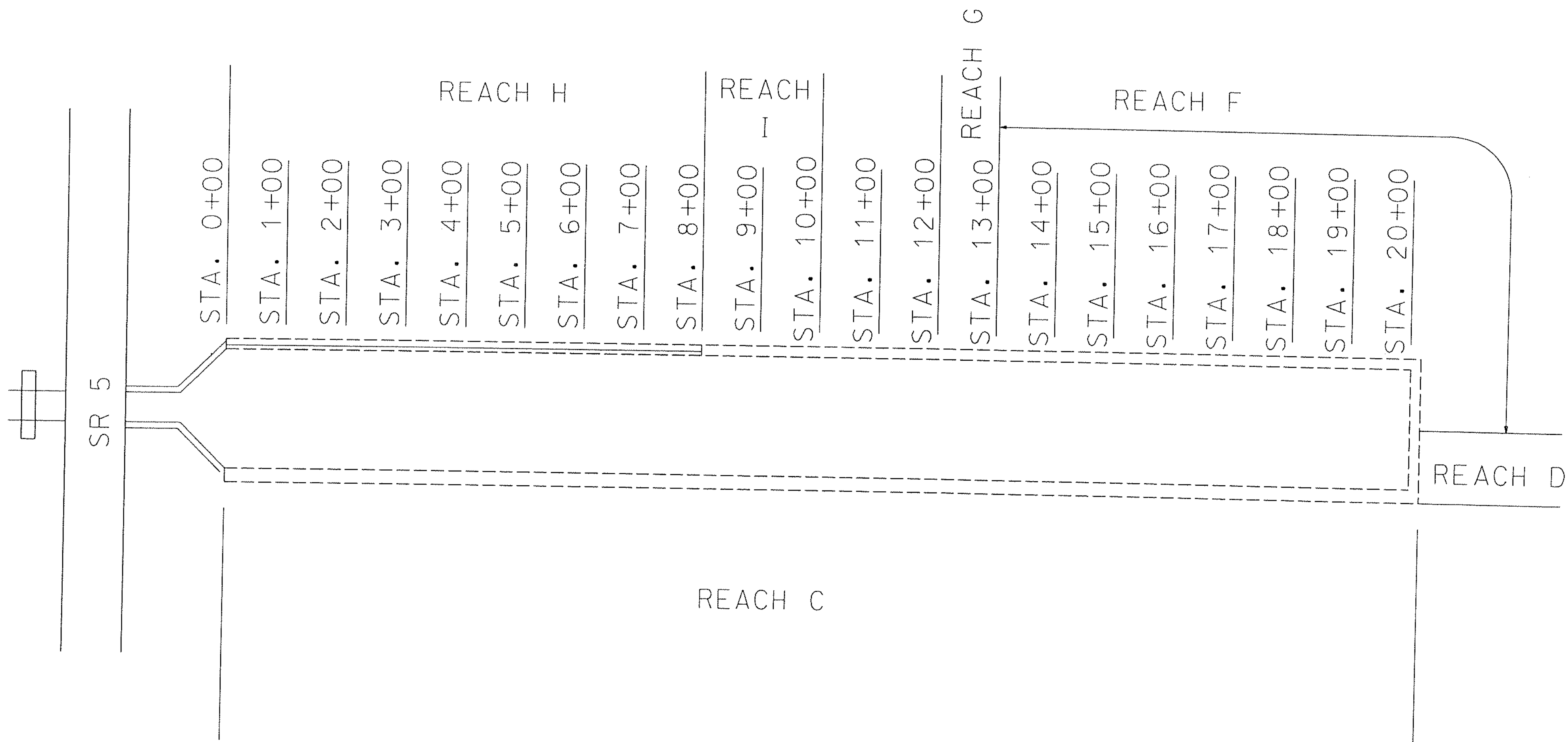
4. "Design Manual for Segmental Retaining Walls" National Concrete Masonry Association, 1993.

5. "Basic Soil Mechanics" R. Whitlow, 1990.

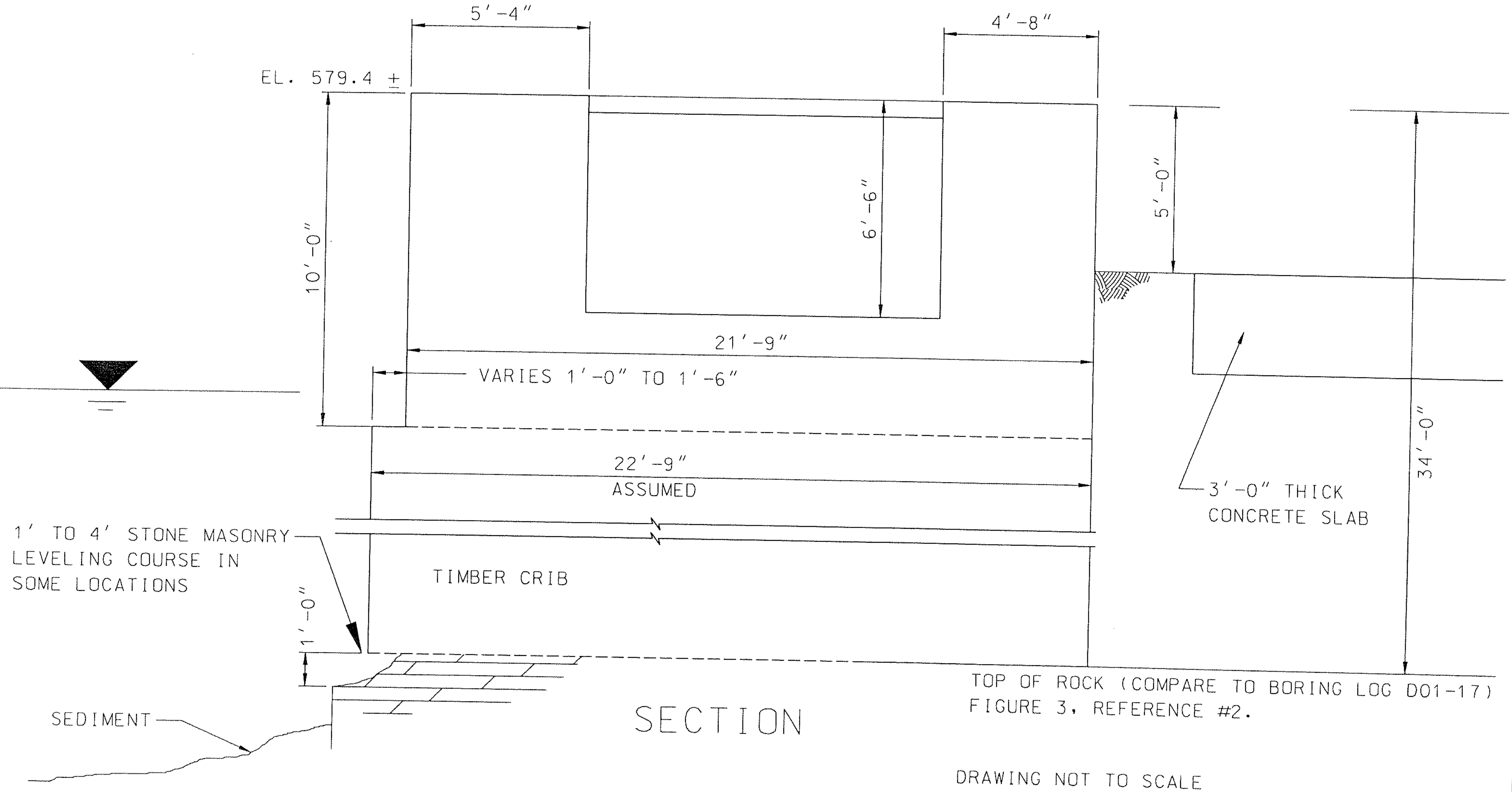
6. "Stability Criteria for Existing Concrete Navigation Structures on Rock Foundations", ETL 1110-2-310, 17 December, 1987.

UNION SHIP CANAL - REACHES & STATIONING



UNION SHIP CANAL - REACH C

(REPRESENTATIVE OF SOUTH WALL STA. 0+00 TO 20+00 +/-)

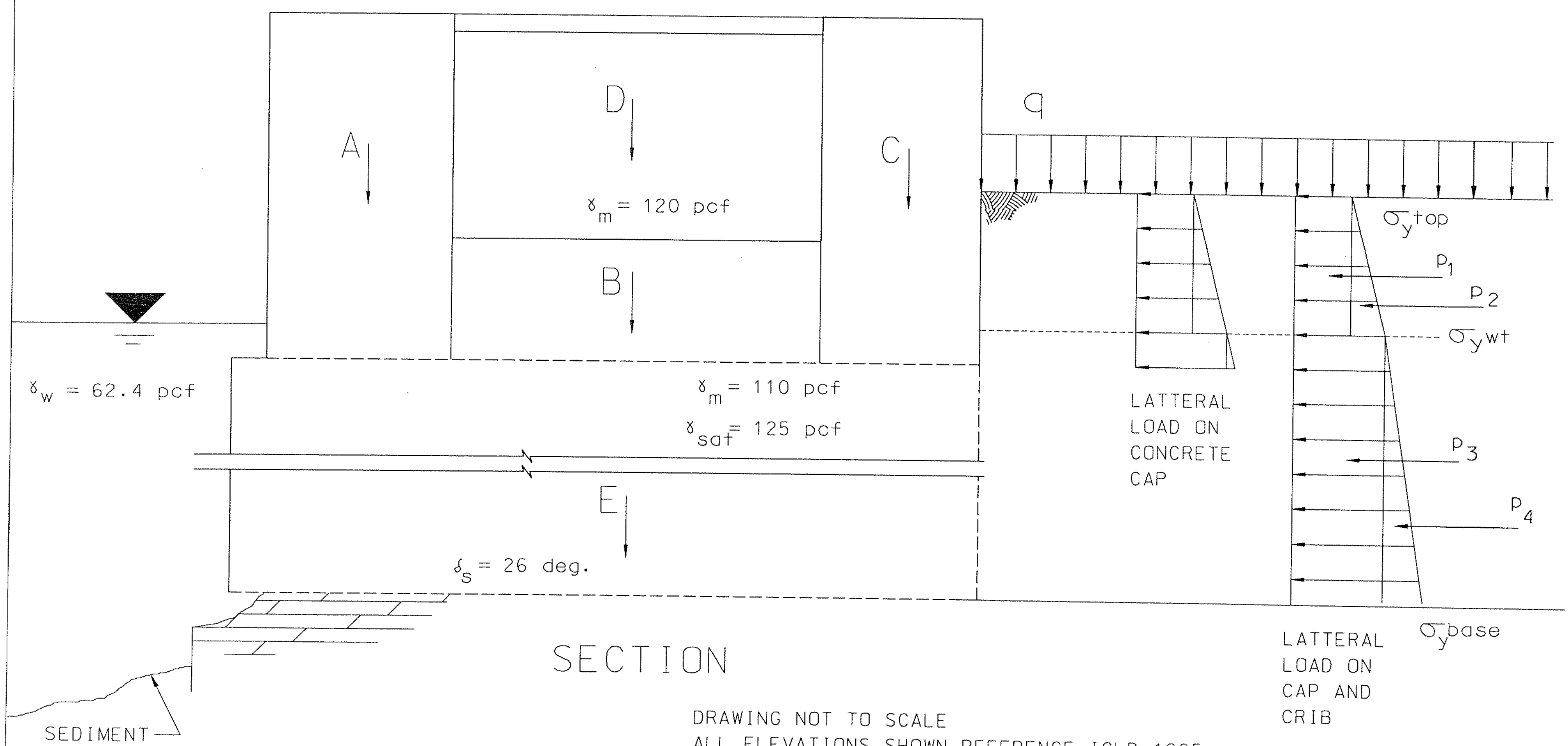


SECTION

DRAWING NOT TO SCALE

ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

UNION SHIP CANAL - REACH C



DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

Stability Anal. of Reach C - Top Cap

Material Properties

Water $\gamma_w := 62.4 \cdot pcf$ Unit Weight of Water

Concrete $\gamma_c := 145 \cdot pcf$ Unit Weight of Concrete

$f'_c := 6000 \cdot psi$ Compressive Strength of Concrete (Reference 1 - Table 1, Page 3)

Soil $\gamma_m := 110 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 3)

Note: Soil parameters used for stability of the top cap are for the course grained fill. Soil parameters used for stability of the entire crib are taken from the more critical values of the course grained fill and the predominantly clay glacial deposits. See reference #2.

$\gamma_{sat} := 127 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 3)

$\phi := 31 \cdot deg$ Angle of Internal Friction (Reference 2 - Figure 3)

$\delta_{conc} := 20 \cdot deg$ Angle of Wall Friction (Concrete) (Reference 2 - Figure 3)

$\delta_{crib} := 27 \cdot deg$ Angle of Wall Friction (Cribbing) (Reference 2 - Figure 3)

$\gamma_e := (\gamma_{sat} - \gamma_w)$ Effective Unit Weight of Submerged Fill $\gamma_e = 64.60 pcf$

$\delta_s := 26 \cdot deg$ Shale (fill) Interface Friction Angle (Reference 2 - Figure 3)

Crib Fill $\gamma_{m\_fill} := 110 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 3)

$\gamma_{sat\_fill} := 125 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 3)

Design Parameters

$SMF := \frac{2}{3}$ Strength Mobilization Factor (Reference 3, Par. 3-13.b)

$\phi_d := atan(SMF \cdot tan(\phi))$ Developed Shear Strength $\phi_d = 21.83 deg$

(Reference 3, eq. 3-10)

$FS_{sliding\_required} := 1.50$ Minimum Sliding Factor of Safety (Reference 3, Table 4-1)

$Pct_{comp\_required} := 100\%$ Minimum Percent Base in Compression (Reference 3, Table 4-1)

Analysis of Top Cap

Structure Dimensions

$B := 21.75 \cdot ft$ Base Width

Structure Elevations

$Top := 5 \cdot ft$ Top of Soil Fill

$Base := 0 \cdot ft$ Base of Structure

$W := 1 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf + 450 \cdot psf$ Surcharge (Snow + Weight of Concrete Slab)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot B$ $U = 1357.20\,plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 550.00\,psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 990.00\,psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 1054.60\,psf$

Latteral Earth Pressure

Horizontal At Rest Earth Pressure Coefficient: $K_O := \left(\tan\left(45 \cdot deg - \frac{\phi_d}{2}\right)\right)^2$ $K_O = 0.46$ (Reference 3, eq. 3-15)

Note: Wall Friction is neglected due to lack of knowledge of back side profile.

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$	Latteral Earth Pressure at Top of Structure	$\sigma_{a\_top} = 251.84\,psf$
$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$	Latteral Earth Pressure at Water Table	$\sigma_{a\_wt} = 453.31\,psf$
$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$	Latteral Earth Pressure at Base of Structure	$\sigma_{a\_bot} = 482.89\,psf$
$P_1 := (\sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_1 = 1007.35\,ft\,psf$
$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P1} = 3.00\,ft$
$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_2 = 402.94\,plf$
$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P2} = 2.33\,ft$
$P_3 := \sigma_{a\_wt} \cdot (W - Base)$	Horizontal Pressure	$P_3 = 453.31\,plf$
$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - Base)\right]$	Resultant Vertical Location	$Y_{P3} = 0.50\,ft$
$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - Base)$	Horizontal Pressure	$P_4 = 14.79\,plf$
$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - Base)$	Resultant Vertical Location	$Y_{P4} = 0.33\,ft$
$P := P_1 + P_2 + P_3 + P_4$	Total Resultant Horizontal Load	$P = 1878.38\,plf$
$Y_P := \frac{\left[(P_1 \cdot Y_{P1}) + (P_2 \cdot Y_{P2}) + (P_3 \cdot Y_{P3}) + (P_4 \cdot Y_{P4})\right]}{P}$	Total Vertical Resultant Location	$Y_P = 2.23\,ft$

Weight of Wall

$W_A := (5.333 \cdot ft) \cdot (10 \cdot ft) \cdot \gamma_c$ $W_A = 7732.85\,plf$

$W_B := (11.75 \cdot ft) \cdot (3.5 \cdot ft) \cdot \gamma_c$ $W_B = 5963.13\,plf$

$W_C := (4.667 \cdot ft) \cdot (10 \cdot ft) \cdot \gamma_c$ $W_C = 6767.15\,plf$

$W_D := (11.75 \cdot ft) \cdot (6.5 \cdot ft) \cdot (120 \cdot pcf)$ $W_D = 9165.00\,plf$

$Weight := W_A + W_B + W_C + W_D$ $Weight = 29628.13\,plf$

Moment Arms of Elements (About Toe)

$Arm_A := \frac{(5.333 \cdot ft)}{2}$ $Arm_A = 2.67\,ft$

$Arm_B := (5.333 \cdot ft) + \frac{(11.75 \cdot ft)}{2}$ $Arm_B = 11.21\,ft$

$Arm_C := 21.75 \cdot ft - \frac{(4.667 \cdot ft)}{2}$ $Arm_C = 19.42\,ft$

$Arm_D := (5.333 \cdot ft) + \frac{(11.75 \cdot ft)}{2}$ $Arm_D = 11.21\,ft$

Resultant Location of Weight

$$X_W := \frac{W_A \cdot Arm_A + W_B \cdot Arm_B + W_C \cdot Arm_C + W_D \cdot Arm_D}{Weight}$$

$X_W = 10.85\,ft$ Horizontal distance from toe to resultant

Resultant Vertical Stress

$N := Weight - U$ $N = 28270.92\,plf$

Sliding Resistance

$\tau_{ult} := N \cdot \tan(\delta_s)$ $\tau_{ult} = 13788.65 \frac{lb}{ft}$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $FS_{sliding\_actual} = 7.34$ > $FS_{sliding\_required} = 1.50$ O.K.

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2}\right)}{N}$$ $X_N = 10.70\,ft$

$$e := \left| X_N - \frac{B}{2} \right|$$ $e = 0.17\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B}\right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 100.00\,\%$ base in compression > $Pct_{comp\_required} = 100.00\,\%$

Stability Anal. of Reach C - Crib

Material Properties

Water $\gamma_w := 62.4 \cdot pcf$ Unit Weight of Water

Concrete $\gamma_c := 145 \cdot pcf$ Unit Weight of Concrete

$f'_c := 6000 \cdot psi$ Compressive Strength of Concrete (Reference 1 - Table 1, Page 3)

Soil Note: Because of anticipated future use of area surrounding the canal, long term strengths can be assumed for the cohesive materials. Assume a single soil layer, choosing the more conservative value listed for the fill and the clay glacial deposits, for the parameters below:

Note: Soil parameters used for stability of the top cap are for the course grained fill. Soil parameters used for stability of the entire crib are taken from the more critical values of the course grained fill and the predominantly clay glacial deposits. See reference #2.

$\gamma_m := 110 \cdot pcf$	Moist Unit Weight (Fill)	(Reference 2 - Figure 3)
$\gamma_{sat} := 127 \cdot pcf$	Saturated Unit Weight (Fill)	(Reference 2 - Figure 3)
$\phi := 29 \cdot deg$	Angle of Internal Friction (Glacial)	(Reference 2 - Figure 3)
$\delta_{conc} := 20 \cdot deg$	Angle of Wall Friction (Concrete)	(Reference 2 - Figure 3)
$\delta_{crib} := 27 \cdot deg$	Angle of Wall Friction (Cribbing)	(Reference 2 - Figure 3)
$\gamma_e := (\gamma_{sat} - \gamma_w)$	Effective Unit Weight of Submerged Soil	$\gamma_e = 64.60 pcf$
$\delta_s := 26 \cdot deg$	Shale (foundation) Interface Friction Angle	(Reference 2 - Figure 3)

Crib Fill

$\gamma_{m\_fill} := 110 \cdot pcf$	Moist Unit Weight	(Reference 2 - Figure 3)
$\gamma_{sat\_fill} := 127 \cdot pcf$	Saturated Unit Weight	(Reference 2 - Figure 3)

Design Parameters

$SMF := \frac{2}{3}$	Strength Mobilization Factor	(Reference 3, Par. 3-13.b)
$\phi_d := atan(SMF \cdot tan(\phi))$	Developed Shear Strength	$\phi_d = 20.28 deg$ (Reference 3, eq. 3-10)
$FS_{sliding\_required} := 1.50$	Minimum Sliding Factor of Safety	(Reference 3, Table 4-1)
$Pct_{comp\_required} := 100\%$	Minimum Percent Base in Compression	(Reference 3, Table 4-1)

Analysis of Crib

Structure Dimensions

$B := 21.75 \cdot ft$ Base Width

1'-0" deducted for undermining.

$Undermine := 1 \cdot ft$

Structure Elevations

$Top := 29 \cdot ft$ Top of Backfill (Reference 1, Boring Log D01-17)

$Base := 0 \cdot ft$ Base of Structure

$W := 25 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf + 450 \cdot psf$ Surcharge (Snow Load + Weight of Concrete Slab)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot (B + Undermine)$ $U = 35490.00 \, plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 550.00 \, psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 990.00 \, psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 2605.00 \, psf$

Latteral Earth Pressure

Horizontal At Rest Earth Pressure Coefficient: $K_O := \left(\tan\left(45 \cdot deg - \frac{\phi_d}{2} \right) \right)^2$ $K_O = 0.49$ (Reference 3, eq. 3-15)

Note: Wall Friction is neglected due to irregular profile.

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$	Latteral Earth Pressure at Top of Structure	$\sigma_{a\_top} = 266.85\,\text{psf}$
$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$	Latteral Earth Pressure at Water Table	$\sigma_{a\_wt} = 480.34\,\text{psf}$
$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$	Latteral Earth Pressure at Base of Structure	$\sigma_{a\_bot} = 1263.92\,\text{psf}$
$P_1 := (\sigma_{a\_top}) \cdot (\text{Top} - W)$	Horizontal Pressure	$P_1 = 1067.42\,\text{ft psf}$
$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (\text{Top} - W)\right]$	Resultant Vertical Location	$Y_{P1} = 27.00\,\text{ft}$
$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (\text{Top} - W)$	Horizontal Pressure	$P_2 = 426.97\,\text{plf}$
$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (\text{Top} - W)\right]$	Resultant Vertical Location	$Y_{P2} = 26.33\,\text{ft}$
$P_3 := \sigma_{a\_wt} \cdot (W - \text{Base})$	Horizontal Pressure	$P_3 = 12008.46\,\text{plf}$
$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - \text{Base})\right]$	Resultant Vertical Location	$Y_{P3} = 12.50\,\text{ft}$
$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - \text{Base})$	Horizontal Pressure	$P_4 = 9794.78\,\text{plf}$
$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - \text{Base})$	Resultant Vertical Location	$Y_{P4} = 8.33\,\text{ft}$
$P := P_1 + P_2 + P_3 + P_4$	Total Resultant Horizontal Load	$P = 23297.62\,\text{plf}$
$Y_P := \dfrac{\left[(P_1 \cdot Y_{P1}) + (P_2 \cdot Y_{P2}) + (P_3 \cdot Y_{P3}) + (P_4 \cdot Y_{P4})\right]}{P}$	Total Vertical Resultant Location	$Y_P = 11.67\,\text{ft}$

Weight of Wall

$W_A := (5.333 \cdot ft) \cdot (10 \cdot ft) \cdot \gamma_c$ $\quad W_A = 7732.85\,plf$

$W_B := (11.75 \cdot ft) \cdot (3.5 \cdot ft) \cdot \gamma_c$ $\quad W_B = 5963.13\,plf$

$W_C := (4.667 \cdot ft) \cdot (10 \cdot ft) \cdot \gamma_c$ $\quad W_C = 6767.15\,plf$

$W_D := (11.75 \cdot ft) \cdot (6.5 \cdot ft) \cdot (120 \cdot pcf)$ $\quad W_D = 9165.00\,plf$

$W_E := (22.75 \cdot ft) \cdot (24 \cdot ft) \cdot \gamma_{sat\_fill}$ $\quad W_E = 69342.00\,plf$

$Weight := W_A + W_B + W_C + W_D + W_E$ $\quad Weight = 98970.13\,plf$

Moment Arms of Elements (About Toe)

$Arm_A := \frac{(5.333 \cdot ft)}{2}$ $\quad Arm_A = 2.67\,ft$

Note: Moment arms are the same as for the analysis of just the cap. Add 1' for the offset of the concrete from the face of the timber crib, subtract 1' for the undermining...no net change.

$Arm_B := (5.333 \cdot ft) + \frac{(11.75 \cdot ft)}{2}$ $\quad Arm_B = 11.21\,ft$

$Arm_C := 21.75 \cdot ft - \frac{(4.667 \cdot ft)}{2}$ $\quad Arm_C = 19.42\,ft$

$Arm_D := (5.333 \cdot ft) + \frac{(11.75 \cdot ft)}{2}$ $\quad Arm_D = 11.21\,ft$

$Arm_E := \frac{(22.75 \cdot ft)}{2} - 1 \cdot ft$ $\quad Arm_E = 10.38\,ft$ Subtract 1' due to undermining.

Resultant Location of Weight

$$X_W := \frac{W_A \cdot Arm_A + W_B \cdot Arm_B + W_C \cdot Arm_C + W_D \cdot Arm_D + W_E \cdot Arm_E}{Weight}$$

$X_W = 10.52\,ft$ Horizontal distance from toe to resultant

Resultant Vertical Stress

$N := Weight - U$ $\quad N = 63480.13\,plf$

Sliding Resistance

$\tau_{ult} := N \cdot \tan(\delta_s)$

$\tau_{ult} = 30961.33 \frac{lb}{ft}$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$

$FS_{sliding\_actual} = 1.33$ < $FS_{sliding\_required} = 1.50$ **FAILS**

Note: Though the required factor of safety is not met for a new structure and normal load condition, the factor of safety for an unusual condition is met (1.33). Assumptions made for the analysis are conservative, i.e. analysis is for tallest portion of the wall, any passive soil has been neglected, most conservative value for each of the soil parameters was used, snow load does not occur over most of the year but is considered a normal load condition. As there is no evidence of instability in the wall (visible misalignment, rotation or translation), for the above reasons, the stability of the wall is considered adequate for the anticipated future uses and loads by engineering judgement.

Resultant location (from center of base)

$$X_N := \frac{\left(\mathrm{Weight} \cdot X_W - P \cdot Y_P - U \cdot \frac{B + \mathrm{Undermine}}{2}\right)}{N}$$

$X_N = 5.76\,\mathrm{ft}$

$$e := \left| X_N - \frac{B}{2} \right|$$

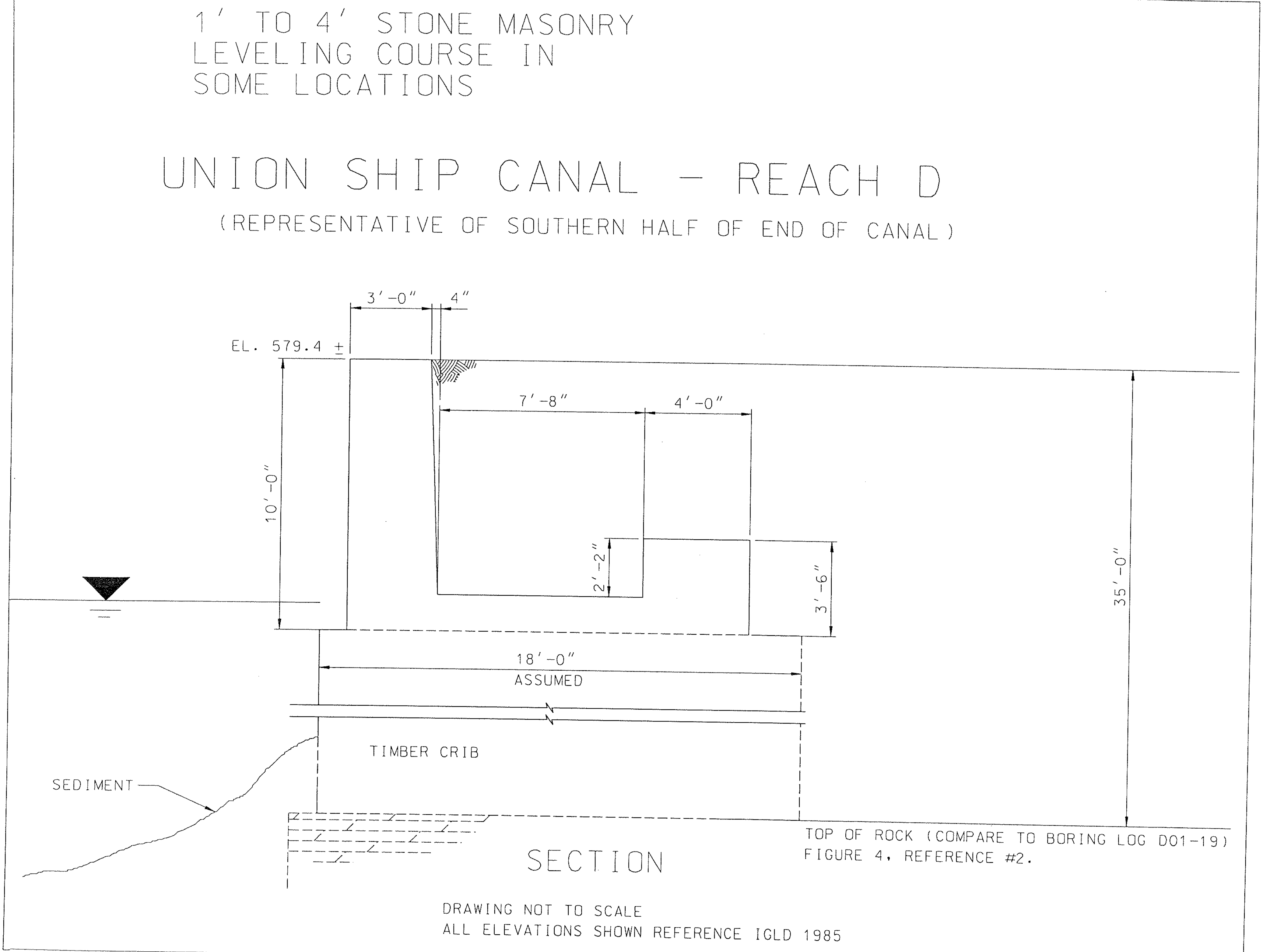
$e = 5.12\,\mathrm{ft}$

$$Pct_{comp\_actual} := \mathrm{if}\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B} \right]$$

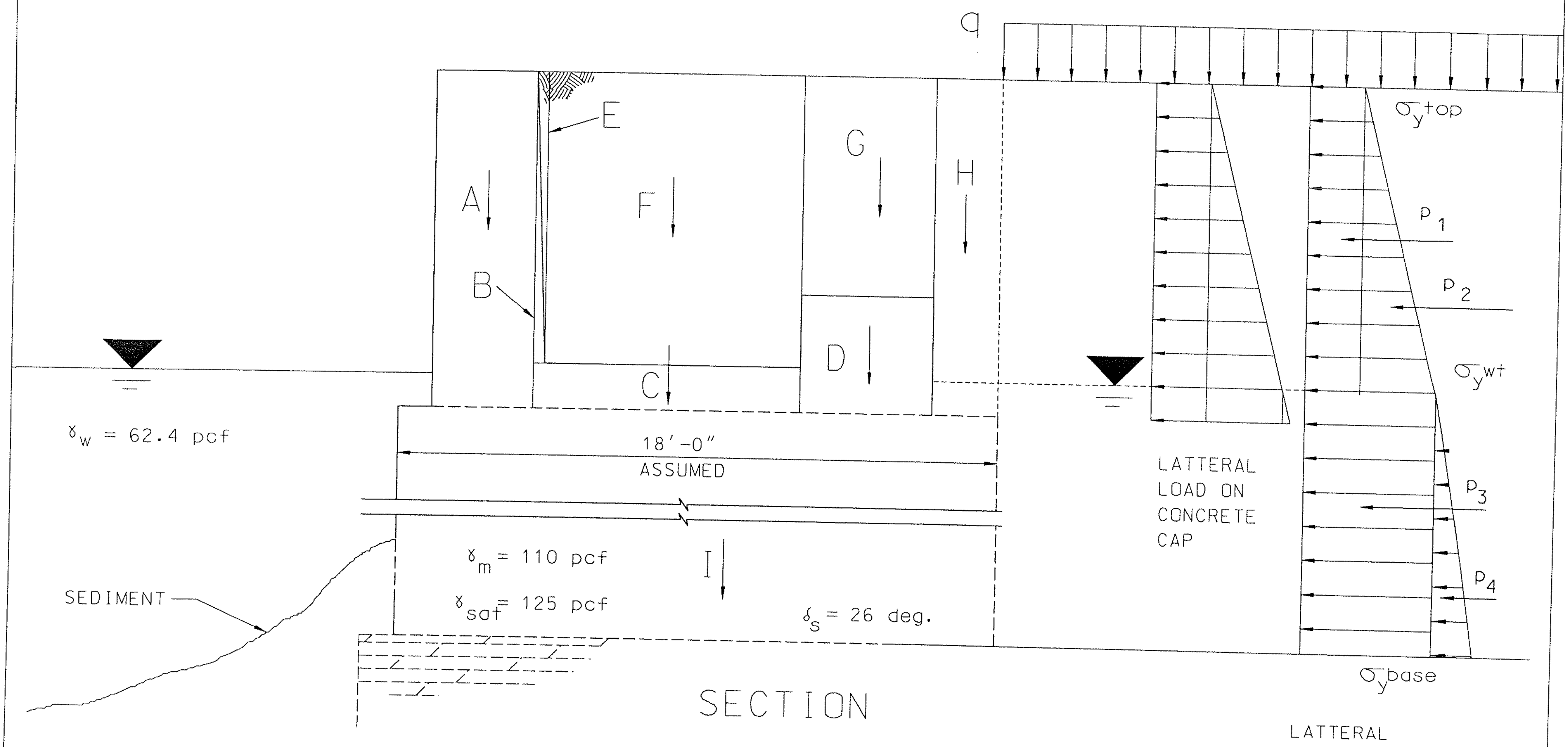
Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 79.42\,\%$ base in compression < $Pct_{comp\_required} = 100.00\,\%$ **FAILS**

Note: Though the required percent base in compression is not met for a new structure and normal load condition, the percent base in compression for an unusual condition is met (75%). Additionally, the percent base in compression is met for a normal load case per Reference #6 (75%). Assumptions made for the analysis are conservative, i.e. analysis is for tallest portion of the wall, any passive soil has been neglected, most conservative value for each of the soil parameters was used, snow load does not occur over most of the year but is considered a normal load condition. As there is no evidence of instability in the wall (visible misalignment, rotation or translation), for the above reasons, the stability of the wall is considered adequate for the anticipated future uses and loads by engineering judgement.


1' TO 4' STONE MASONRY
LEVELING COURSE IN
SOME LOCATIONS
UNION SHIP CANAL - REACH D
(REPRESENTATIVE OF SOUTHERN HALF OF END OF CANAL)
3'-0"
4"
EL. 579.4 ±
7'-8"
4'-0"
10'-0"
2'-2"
3'-6"
35'-0"
18'-0"
ASSUMED
TIMBER CRIB
SEDIMENT
TOP OF ROCK (COMPARE TO BORING LOG D01-19)
FIGURE 4, REFERENCE #2.
SECTION
DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

UNION SHIP CANAL - REACH D



SECTION

DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

Stability Anal. of Reach D - Top Cap

Material Properties

Water

$\gamma_w := 62.4 \cdot pcf$ Unit Weight of Water

Concrete

$\gamma_c := 145 \cdot pcf$ Unit Weight of Concrete

$f'_c := 6000 \cdot psi$ Compressive Strength of Concrete (Reference 1 - Table 1, Page 3)

Soil

$\gamma_m := 100 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 4)

$\gamma_{sat} := 120 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 4)

$\phi := 28 \cdot deg$ Angle of Internal Friction (Reference 2 - Figure 4)

$\delta_{conc} := 20 \cdot deg$ Angle of Wall Friction (Concrete) (Reference 2 - Figure 4)

$\delta_{crib} := 26 \cdot deg$ Angle of Wall Friction (Cribbing) (Reference 2 - Figure 4)

$\gamma_e := (\gamma_{sat} - \gamma_w)$ Effective Unit Weight of Submerged Fill $\gamma_e = 57.60 pcf$

$\delta_s := 26 \cdot deg$ Shale (fill) Interface Friction Angle (Reference 2 - Figure 4)

Crib Fill

$\gamma_{m\_fill} := 110 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 4)

$\gamma_{sat\_fill} := 125 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 4)

Design Parameters

$SMF := \frac{2}{3}$ Strength Mobilization Factor (Reference 3, Par. 3-13.b)

$\phi_d := atan(SMF \cdot tan(\phi))$ Developed Shear Strength $\phi_d = 19.52 deg$

(Reference 3, eq. 3-10)

$FS_{sliding\_required} := 1.5$ Minimum Sliding Factor of Safety (Reference 3, Table 4-1)

$Pct_{comp\_required} := 100\%$ Minimum Percent Base in Compression (Reference 3, Table 4-1)

Analysis of Top Cap

Structure Dimensions

$B := 15 \cdot ft$ Base Width

Structure Elevations

$Top := 10 \cdot ft$ Top of Soil Fill

$Base := 0 \cdot ft$ Base of Structure

$W := 1 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf$ Surcharge (Snow)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot B$ $U = 936.00\,plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 100.00\,psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 1000.00\,psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 1057.60\,psf$

Latteral Earth Pressure

Horizontal At Rest Earth Pressure Coefficient: $K_O := \left(\tan\left(45 \cdot deg - \frac{\phi_d}{2}\right)\right)^2$ $K_O = 0.50$ (Reference 3, eq. 3-15)

Note: Wall Friction is neglected due to irregular profile.

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$	Latteral Earth Pressure at Top of Structure	$\sigma_{a\_top} = 49.91\,\text{psf}$
$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$	Latteral Earth Pressure at Water Table	$\sigma_{a\_wt} = 499.13\,\text{psf}$
$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$	Latteral Earth Pressure at Base of Structure	$\sigma_{a\_bot} = 527.88\,\text{psf}$
$P_1 := (\sigma_{a\_top}) \cdot (\text{Top} - W)$	Horizontal Pressure	$P_1 = 449.22\,\text{ft psf}$
$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (\text{Top} - W)\right]$	Resultant Vertical Location	$Y_{P1} = 5.50\,\text{ft}$
$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (\text{Top} - W)$	Horizontal Pressure	$P_2 = 2021.49\,\text{plf}$
$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (\text{Top} - W)\right]$	Resultant Vertical Location	$Y_{P2} = 4.00\,\text{ft}$
$P_3 := \sigma_{a\_wt} \cdot (W - \text{Base})$	Horizontal Pressure	$P_3 = 499.13\,\text{plf}$
$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - \text{Base})\right]$	Resultant Vertical Location	$Y_{P3} = 0.50\,\text{ft}$
$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - \text{Base})$	Horizontal Pressure	$P_4 = 14.38\,\text{plf}$
$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - \text{Base})$	Resultant Vertical Location	$Y_{P4} = 0.33\,\text{ft}$
$P := P_1 + P_2 + P_3 + P_4$	Total Resultant Horizontal Load	$P = 2984.22\,\text{plf}$
$Y_P := \frac{\left[(P_1 \cdot Y_{P1}) + (P_2 \cdot Y_{P2}) + (P_3 \cdot Y_{P3}) + (P_4 \cdot Y_{P4})\right]}{P}$	Total Vertical Resultant Location	$Y_P = 3.62\,\text{ft}$

Weight of Wall

$W_A := (3\cdot ft)\cdot(10\cdot ft)\cdot\gamma_c$ $\quad W_A = 4350.00\,plf$

$W_B := \left(\frac{1}{2}\right)(0.333\cdot ft)\cdot(8.667\cdot ft)\cdot\gamma_c$ $\quad W_B = 209.24\,plf$

$W_C := (8\cdot ft)\cdot(1.333\cdot ft)\cdot\gamma_c$ $\quad W_C = 1546.28\,plf$

$W_D := (3.5\cdot ft)\cdot(4\cdot ft)\cdot\gamma_c$ $\quad W_D = 2030.00\,plf$

$W_E := \left(\frac{1}{2}\right)(0.333\cdot ft)\cdot(8.667\cdot ft)\cdot\gamma_m$ $\quad W_E = 144.31\,plf$

$W_F := (7.667\cdot ft)\cdot(8.667\cdot ft)\cdot\gamma_m$ $\quad W_F = 6644.99\,plf$

$W_G := (4\cdot ft)\cdot(6.5\cdot ft)\cdot\gamma_m$ $\quad W_G = 2600.00\,plf$

$Weight := W_A + W_B + W_C + W_D + W_E + W_F + W_G$ $\quad Weight = 17524.82\,plf$

Moment Arms of Elements (About Toe)

$Arm_A := \frac{(3\cdot ft)}{2}$ $\quad Arm_A = 1.50\,ft$

$Arm_B := (3\cdot ft) + \frac{(0.333\cdot ft)}{3}$ $\quad Arm_B = 3.11\,ft$

$Arm_C := 3\cdot ft + \frac{(8\cdot ft)}{2}$ $\quad Arm_C = 7.00\,ft$

$Arm_D := (11\cdot ft) + \frac{(4\cdot ft)}{2}$ $\quad Arm_D = 13.00\,ft$

$Arm_E := (3\cdot ft) + \frac{2(0.333\cdot ft)}{3}$ $\quad Arm_E = 3.22\,ft$

$Arm_F := (3.333\cdot ft) + \frac{(7.667\cdot ft)}{2}$ $\quad Arm_F = 7.17\,ft$

$Arm_G := (11\cdot ft) + \frac{(4\cdot ft)}{2}$ $\quad Arm_G = 13.00\,ft$

Resultant Location of Weight

$$X_W := \frac{W_A \cdot Arm_A + W_B \cdot Arm_B + W_C \cdot Arm_C + W_D \cdot Arm_D + W_E \cdot Arm_E + W_F \cdot Arm_F + W_G \cdot Arm_G}{Weight}$$

$X_W = 7.21\,ft$ Horizontal distance from toe to resultant

Resultant Vertical Stress

$N := Weight - U$ $N = 16588.82\,plf$

Sliding Resistance

$\tau_{ult} := N \cdot \tan(\delta_s)$ $\tau_{ult} = 8090.91\,\frac{lb}{ft}$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $FS_{sliding\_actual} = 2.71$ > $FS_{sliding\_required} = 1.50$ O.K.

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2}\right)}{N}$$ $X_N = 6.54\,ft$

$e := \left| X_N - \frac{B}{2} \right|$ $e = 0.96\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B}\right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 100.00\,\%$ base in compression > $Pct_{comp\_required} = 100.00\,\%$

Stability Anal. of Reach D - Crib

Material Properties

Water

$\gamma_w := 62.4 \cdot pcf$ Unit Weight of Water

Concrete

$\gamma_c := 145 \cdot pcf$ Unit Weight of Concrete

$f'_c := 6000 \cdot psi$ Compressive Strength of Concrete (Reference 1 - Table 1, Page 3)

Soil

$\gamma_m := 100 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 4)

$\gamma_{sat} := 120 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 4)

$\phi := 28 \cdot deg$ Angle of Internal Friction (Reference 2 - Figure 4)

$\delta_{conc} := 20 \cdot deg$ Angle of Wall Friction (Concrete) (Reference 2 - Figure 4)

$\delta_{crib} := 26 \cdot deg$ Angle of Wall Friction (Cribbing) (Reference 2 - Figure 4)

$\gamma_e := (\gamma_{sat} - \gamma_w)$ Effective Unit Weight of Submerged Fill $\gamma_e = 57.60 pcf$

$\delta_s := 26 \cdot deg$ Shale (fill) Interface Friction Angle (Reference 2 - Figure 4)

Crib Fill

$\gamma_{m\_fill} := 110 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 4)

$\gamma_{sat\_fill} := 125 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 4)

Design Parameters

$SMF := \frac{2}{3}$ Strength Mobilization Factor (Reference 3, Par. 3-13.b)

$\phi_d := \operatorname{atan}(SMF \cdot \tan(\phi))$ Developed Shear Strength $\phi_d = 19.52 deg$

(Reference 3, eq. 3-10)

$FS_{sliding\_required} := 1.50$ Minimum Sliding Factor of Safety (Reference 3, Table 4-1)

$Pct_{comp\_required} := 100\%$ Minimum Percent Base in Compression (Reference 3, Table 4-1)

Analysis of Crib

Structure Dimensions

$B := 18 \cdot ft$ Base Width

Structure Elevations

$Top := 35 \cdot ft$ Top of Soil Fill

$Base := 0 \cdot ft$ Base of Structure

$W := 26 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf$ Surcharge (Snow)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot B$ $U = 29203.20\, plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 100.00\, psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 1000.00\, psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 2497.60\, psf$

Latteral Earth Pressure

Horizontal At Rest Earth Pressure Coefficient: $K_O := \left(\tan\left(45 \cdot deg - \frac{\phi d}{2}\right)\right)^2$ $K_O = 0.50$ (Reference 3, eq. 3-15)

Note: Wall Friction is neglected due to irregular profile.

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$ — Latteral Earth Pressure at Top of Structure — $\sigma_{a\_top} = 49.91\,psf$

$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$ — Latteral Earth Pressure at Water Table — $\sigma_{a\_wt} = 499.13\,psf$

$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$ — Latteral Earth Pressure at Base of Structure — $\sigma_{a\_bot} = 1246.64\,psf$

$P_1 := (\sigma_{a\_top}) \cdot (Top - W)$ — Horizontal Pressure — $P_1 = 449.22\,ft\,psf$

$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (Top - W)\right]$ — Resultant Vertical Location — $Y_{P1} = 30.50\,ft$

$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (Top - W)$ — Horizontal Pressure — $P_2 = 2021.49\,plf$

$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (Top - W)\right]$ — Resultant Vertical Location — $Y_{P2} = 29.00\,ft$

$P_3 := \sigma_{a\_wt} \cdot (W - Base)$ — Horizontal Pressure — $P_3 = 12977.48\,plf$

$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - Base)\right]$ — Resultant Vertical Location — $Y_{P3} = 13.00\,ft$

$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - Base)$ — Horizontal Pressure — $P_4 = 9717.54\,plf$

$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - Base)$ — Resultant Vertical Location — $Y_{P4} = 8.67\,ft$

$P := P_1 + P_2 + P_3 + P_4$ — Total Resultant Horizontal Load — $P = 25165.73\,plf$

$$Y_P := \frac{\left[(P_1 \cdot Y_{P1}) + (P_2 \cdot Y_{P2}) + (P_3 \cdot Y_{P3}) + (P_4 \cdot Y_{P4})\right]}{P}$$

$Y_P = 12.92\,ft$

Total Vertical Resultant Location

Weight of Wall

$W_A := (3 \cdot ft) \cdot (10 \cdot ft) \cdot \gamma_c$ $\quad W_A = 4350.00 \, plf$

$W_B := \left(\frac{1}{2}\right)(0.333 \cdot ft) \cdot (8.667 \cdot ft) \cdot \gamma_c$ $\quad W_B = 209.24 \, plf$

$W_C := (8 \cdot ft) \cdot (1.333 \cdot ft) \cdot \gamma_c$ $\quad W_C = 1546.28 \, plf$

$W_D := (3.5 \cdot ft) \cdot (4 \cdot ft) \cdot \gamma_c$ $\quad W_D = 2030.00 \, plf$

$W_E := \left(\frac{1}{2}\right)(0.333 \cdot ft) \cdot (8.667 \cdot ft) \cdot \gamma_m$ $\quad W_E = 144.31 \, plf$

$W_F := (7.667 \cdot ft) \cdot (8.667 \cdot ft) \cdot \gamma_m$ $\quad W_F = 6644.99 \, plf$

$W_G := (4 \cdot ft) \cdot (6.5 \cdot ft) \cdot \gamma_m$ $\quad W_G = 2600.00 \, plf$

$W_H := (2 \cdot ft) \cdot (9 \cdot ft) \cdot \gamma_m + (2 \cdot ft) \cdot (1 \cdot ft) \cdot \gamma_{sat}$ $\quad W_H = 2040.00 \, plf$

$W_I := (18 \cdot ft) \cdot (25 \cdot ft) \cdot \gamma_{sat\_fill}$ $\quad W_I = 56250.00 \, plf$

$Weight := W_A + W_B + W_C + W_D + W_E + W_F + W_G + W_H + W_I$ $\quad Weight = 75814.82 \, plf$

Moment Arms of Elements (About Toe)

$Arm_A := \frac{(3 \cdot ft)}{2} + 1 \cdot ft$ $\quad Arm_A = 2.50 \, ft$

$Arm_B := (3 \cdot ft) + \frac{(0.333 \cdot ft)}{3} + 1 \cdot ft$ $\quad Arm_B = 4.11 \, ft$

$Arm_C := 3 \cdot ft + \frac{(8 \cdot ft)}{2} + 1 \cdot ft$ $\quad Arm_C = 8.00 \, ft$

$Arm_D := (11 \cdot ft) + \frac{(4 \cdot ft)}{2} + 1 \cdot ft$ $\quad Arm_D = 14.00 \, ft$

$Arm_E := (3 \cdot ft) + \frac{2(0.333 \cdot ft)}{3} + 1 \cdot ft$ $\quad Arm_E = 4.22 \, ft$

$Arm_F := (3.333 \cdot ft) + \frac{(7.667 \cdot ft)}{2} + 1 \cdot ft$ $\quad Arm_F = 8.17 \, ft$

$Arm_G := (11 \cdot ft) + \frac{(4 \cdot ft)}{2} + 1 \cdot ft$ $\quad Arm_G = 14.00 \, ft$

$Arm_H := (18 \cdot ft) - \frac{(2 \cdot ft)}{2}$ $\qquad Arm_H = 17.00\,ft$

$Arm_I := \frac{(18 \cdot ft)}{2}$ $\qquad Arm_I = 9.00\,ft$

Resultant Location of Weight

$$X_W := \frac{W_A \cdot Arm_A + W_B \cdot Arm_B + W_C \cdot Arm_C + W_D \cdot Arm_D + W_E \cdot Arm_E + W_F \cdot Arm_F + W_G \cdot Arm_G + W_H \cdot Arm_H + W_I \cdot Arm_I}{Weight}$$

$X_W = 9.03\,ft$ Horizontal distance from toe to resultant

Resultant Vertical Stress

$N := Weight - U$ $\qquad N = 46611.62\,plf$

Sliding Resistance

$\tau_{ult} := N \cdot \tan(\delta_s)$ $\qquad \tau_{ult} = 22734.00\,\frac{lb}{ft}$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $\qquad FS_{sliding\_actual} = 0.90 \quad < \quad FS_{sliding\_required} = 1.50$ Fails

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2}\right)}{N}$$ $\qquad X_N = 2.07\,ft$

$e := \left| X_N - \frac{B}{2} \right|$ $\qquad e = 6.93\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B} \right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 34.56\%$ base in compression $\quad < \quad Pct_{comp\_required} = 100.00\%$

Account for Resistance of Passive Soil

Note: Sediment was visually seen from above the water surface to extend up to within a few feet of the surface at the east end corners. Soundings most likely were not capable of capturing this due to debris. Average depth of the canal is -20' LWD (EL. 549.2). Top of rock is approximately 544.4, say 5' of passive sediment can be relied upon. Sediment assumed to be very loose silt.

$h_{silt} := 5 \cdot ft$

$\gamma_{e\_silt\_sat} := 115 \cdot pcf - \gamma_w$ $\qquad \gamma_{e\_silt\_sat} = 52.60\,pcf$

$\phi_{silt} := 27 \cdot deg$

Note: SMF is not used for calculating passive earth pressure coefficients per Reference 3, Para. 3-13.b.

Vertical Stress at Base - Passive Side

Vertical Stress at Base: $\sigma_{ybase\_silt} := \gamma_{e\_silt\_sat}\left(h_{silt}\right)$ $\qquad \sigma_{ybase\_silt} = 263.00\,psf$

Horizontal At Rest Earth Pressure Coefficient: $K_p := \left(\tan\left(45 \cdot deg + \frac{\phi_{silt}}{2}\right)\right)^2$ $\qquad K_p = 2.66$ (Reference 3, eq. 3-20)

Note: Wall Friction is neglected due to irregular profile.

Latteral Earth Pressure at Base of Structure: $\sigma_{p\_bot\_silt} := K_p \cdot \left(\sigma_{ybase\_silt}\right)$ $\qquad \sigma_{p\_bot\_silt} = 700.35\,psf$

Horizontal Pressure: $P_{silt} := .5 \cdot \left(\sigma_{p\_bot\_silt}\right) \cdot \left(h_{silt}\right)$ $\qquad P_{silt} = 1750.88\,ft\,psf$

Resultant Vertical Location: $Y_{Psilt} := \frac{\left(h_{silt}\right)}{3}$ $\qquad Y_{Psilt} = 1.67\,ft$

Sliding Factor of Safety: $FS_{sliding\_actual} := \frac{\tau_{ult}}{P - \left(\frac{P_{silt}}{2}\right)}$ $\qquad FS_{sliding\_actual} = 0.94$

$< \quad FS_{sliding\_required} = 1.50$ Fails

Note: Reference 3, Para. 3-8.b limits passive resistance for stability calculations to 1/2 that of calculated passive resistance.

Resultant location (from center of base)

$$X_N := \frac{\left(Weight\cdot X_W - P\cdot Y_P - U\cdot\frac{B}{2} + P_{silt}\cdot Y_{Psilt}\right)}{N}$$ $X_N = 2.14\,ft$

$$e := \left|X_N - \frac{B}{2}\right|$$ $e = 6.86\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3\cdot\frac{\left(\frac{B}{2} - e\right)}{B}\right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 35.60\%$ base in compression < $Pct_{comp\_required} = 100.00\%$

If stabilization methods were implemented, what would be the necessary height of the passive soil?

Note: Assume a stone berm is placed. Stone will be placed on top of silt, silt will not be removed, so silt will act as part of berm. Use parameters of 118 pcf and $\phi = 30$ deg for stone. As both of these parameters are similar to those of the silt, it is conservative to use the parameters for the silt for the design. Determine required slope of berm for stability. Assume maximum allowable height of berm to be at top of crib (25').

$h_{silt} := 25\cdot ft$ $\beta_{berm} := -15\cdot deg$

$\gamma_{e\_silt\_sat} := 115\cdot pcf - \gamma_w$ $\gamma_{e\_silt\_sat} = 52.60\,pcf$

$\phi_{silt} := 27\cdot deg$

Note: SMF is not used for calculating passive earth pressure coefficients per Reference 3, Para. 3-13.b.

Vertical Stress at Base - Passive Side

Vertical Stress at Base: $\sigma_{ybase\_silt} := \gamma_{e\_silt\_sat}\cdot(h_{silt})$ $\sigma_{ybase\_silt} = 1315.00\,psf$

Horizontal At Rest Earth Pressure Coefficient:

$$K_p := \frac{(\cos(\phi_{silt}))^2}{\left(1 - \sqrt{\frac{\sin(\phi_{silt})\cdot\sin(\phi_{silt} + \beta_{berm})}{\cos(\beta_{berm})}}\right)^2}$$

$K_p = 1.68$ (Reference 3, eq. 3-20)

Note: Wall Friction is neglected due to irregular profile.

Latteral Earth Pressure at Base of Structure: $\sigma_{p\_bot\_silt} := K_p\cdot(\sigma_{ybase\_silt})$ $\sigma_{p\_bot\_silt} = 2209.38\,psf$

Horizontal Pressure: $P_{silt} := .5\cdot(\sigma_{p\_bot\_silt})\cdot(h_{silt})$ $P_{silt} = 27617.24\,ft\,psf$

Resultant Vertical Location $Y_{Psilt} := \frac{(h_{silt})}{3}$ $Y_{Psilt} = 8.33\,ft$

Sliding Factor of Safety $FS_{sliding\_actual} := \frac{\tau_{ult}}{P - \left(\frac{P_{silt}}{2}\right)}$ $FS_{sliding\_actual} = 2.00$ O.K.

< $FS_{sliding\_required} = 1.50$

Note: Reference 3, Para. 3-8.b limits passive resistance for stability calculations to 1/2 that of calculated passive resistance.

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2} + \frac{P_{silt}}{2} \cdot Y_{Psilt}\right)}{N}$$ $X_N = 4.54\,ft$

$$e := \left|X_N - \frac{B}{2}\right|$$ $e = 4.46\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B}\right]$$

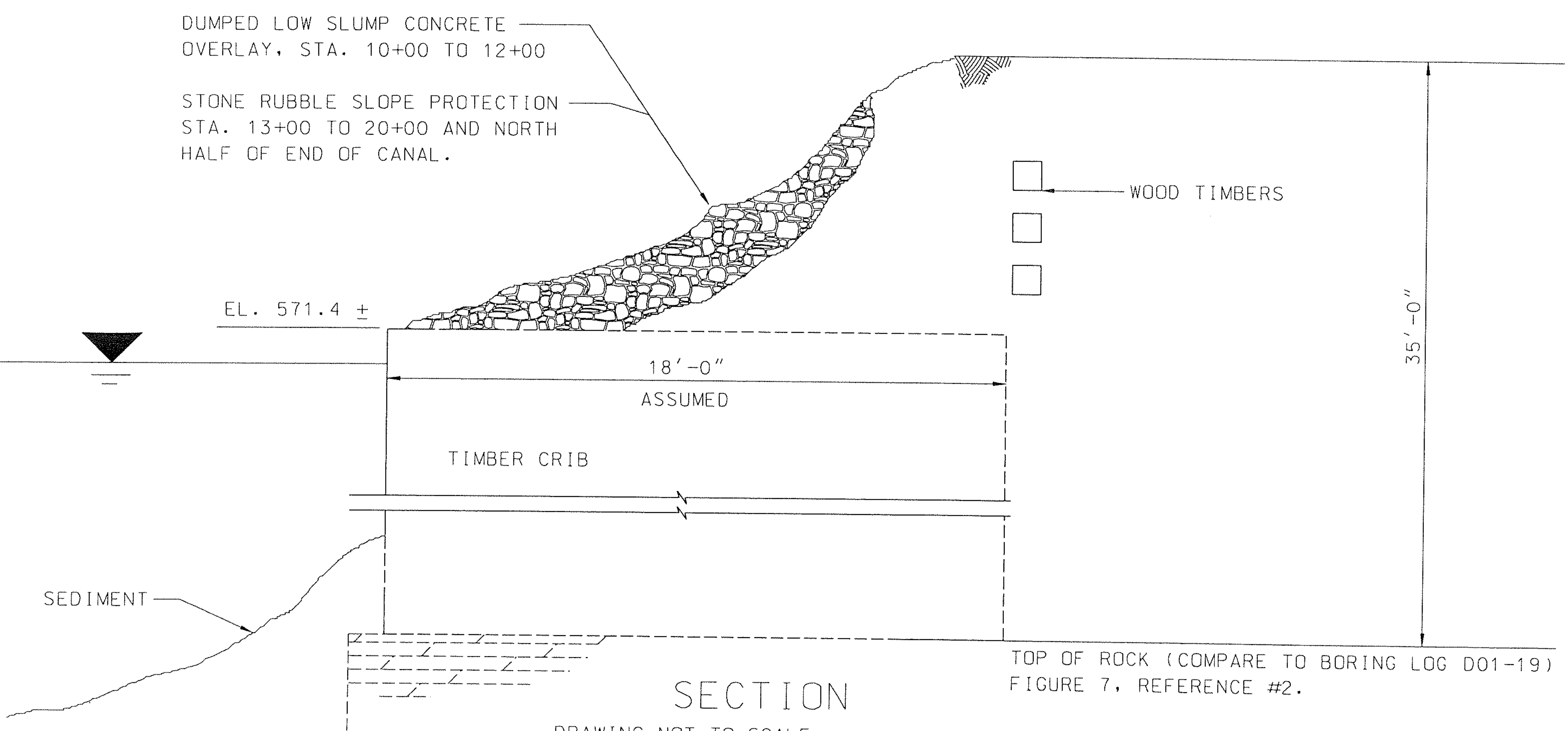
Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 75.70\,\%$ base in compression < $Pct_{comp\_required} = 100.00\,\%$ **<u>FAILS</u>**

Note: With a berm, the factor of safety for sliding meets current design requirements and the percent base in compression meets the requirements for existing structures.

UNION SHIP CANAL - REACH F

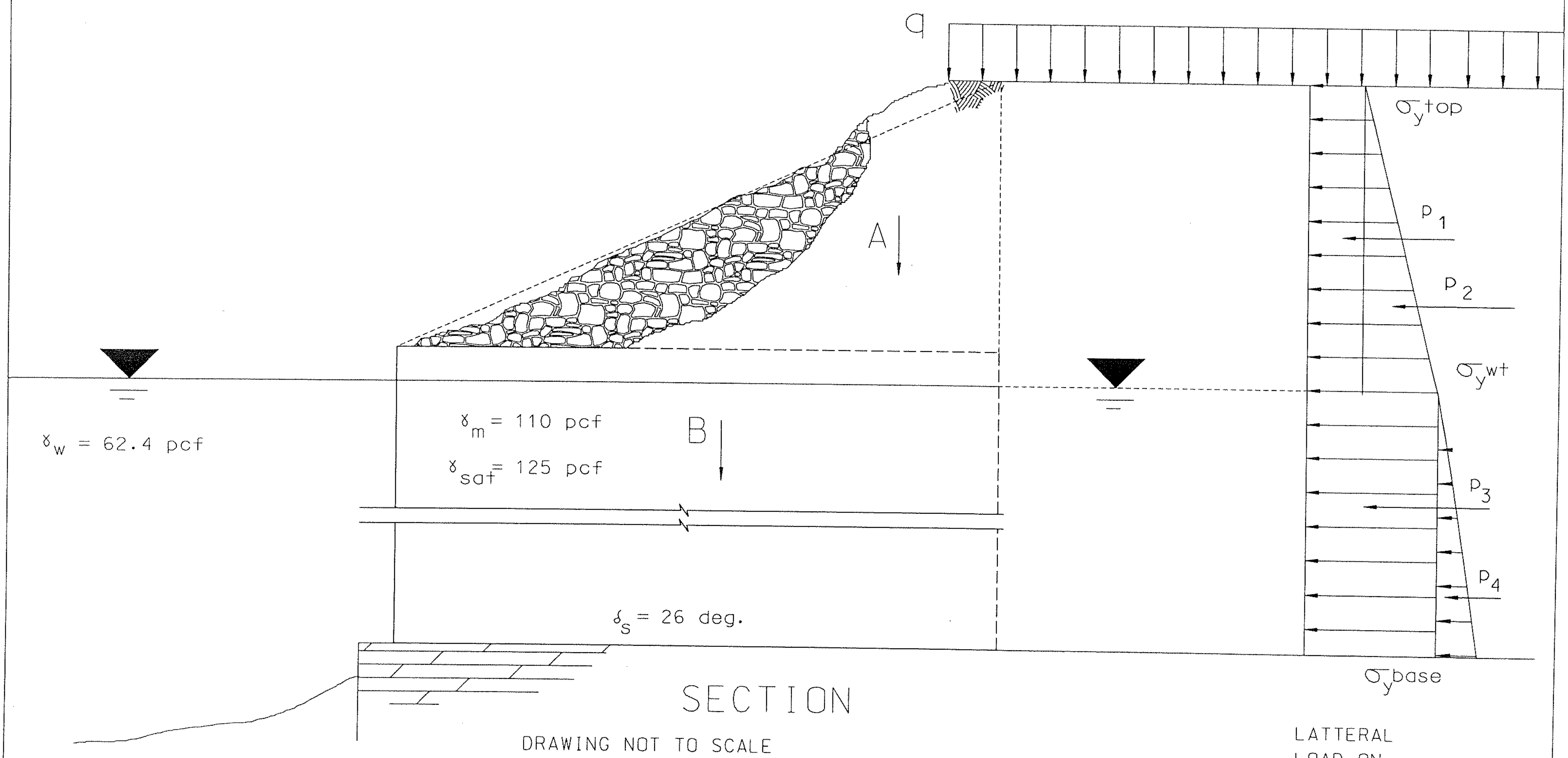
(REPRESENTATIVE OF STA. 10+00 TO 12+00, 13+00 TO 20+00 AND NORTH HALF OF END OF CANAL)



SECTION

DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

UNION SHIP CANAL - REACH F



SECTION

DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

Stability Anal. of Reach F - Crib

Material Properties

Water	$\gamma_w := 62.4 \cdot pcf$	Unit Weight of Water	
Concrete	$\gamma_c := 145 \cdot pcf$	Unit Weight of Concrete	
	$f'_c := 6000 \cdot psi$	Compressive Strength of Concrete	(Reference 1 - Table 1, Page 3)
Soil	$\gamma_m := 110 \cdot pcf$	Moist Unit Weight	(Reference 2 - Figure 7)
	$\gamma_{sat} := 127 \cdot pcf$	Saturated Unit Weight	(Reference 2 - Figure 7)
	$\phi := 29 \cdot deg$	Angle of Internal Friction	(Reference 2 - Figure 7)
	$\delta_{conc} := 20 \cdot deg$	Angle of Wall Friction (Concrete)	(Reference 2 - Figure 7)
	$\delta_{crib} := 26 \cdot deg$	Angle of Wall Friction (Cribbing)	(Reference 2 - Figure 7)
	$\gamma_e := (\gamma_{sat} - \gamma_w)$	Effective Unit Weight of Submerged Fill	$\gamma_e = 64.60 pcf$
	$\delta_s := 26 \cdot deg$	Shale (fill) Interface Friction Angle	(Reference 2 - Figure 7)
Crib Fill	$\gamma_{m\_fill} := 110 \cdot pcf$	Moist Unit Weight	(Reference 2 - Figure 7)
	$\gamma_{sat\_fill} := 125 \cdot pcf$	Saturated Unit Weight	(Reference 2 - Figure 7)

Design Parameters

$SMF := \frac{2}{3}$	Strength Mobilization Factor	(Reference 3, Par. 3-13.b)
$\phi_d := atan(SMF \cdot tan(\phi))$	Developed Shear Strength	$\phi_d = 20.28 deg$ (Reference 3, eq. 3-10)
$FS_{sliding\_required} := 1.50$	Minimum Sliding Factor of Safety	(Reference 3, Table 4-1)
$Pct_{comp\_required} := 100\%$	Minimum Percent Base in Compression	(Reference 3, Table 4-1)

Analysis of Top Cap

Structure Dimensions

$B := 18 \cdot ft$ Base Width

Structure Elevations

$Top := 35 \cdot ft$ Top of Soil Fill

$Base := 0 \cdot ft$ Base of Structure

$W := 26 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf$ Surcharge (Snow)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot B$ $U = 29203.20 \, plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 100.00 \, psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 1090.00 \, psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 2769.60 \, psf$

Latteral Earth Pressure

Horizontal At Rest Earth Pressure Coefficient: $K_O := \left(\tan\left(45 \cdot deg - \frac{\phi_d}{2} \right) \right)^2$ $K_O = 0.49$ (Reference 3, eq. 3-15)

Note: Wall Friction is neglected due to irregular profile.

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$	Latteral Earth Pressure at Top of Structure	$\sigma_{a\_top} = 48.52\,psf$
$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$	Latteral Earth Pressure at Water Table	$\sigma_{a\_wt} = 528.86\,psf$
$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$	Latteral Earth Pressure at Base of Structure	$\sigma_{a\_bot} = 1343.78\,psf$
$P_1 := (\sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_1 = 436.67\,ft\,psf$
$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P1} = 30.50\,ft$
$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_2 = 2161.52\,plf$
$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P2} = 29.00\,ft$
$P_3 := \sigma_{a\_wt} \cdot (W - Base)$	Horizontal Pressure	$P_3 = 13750.29\,plf$
$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - Base)\right]$	Resultant Vertical Location	$Y_{P3} = 13.00\,ft$
$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - Base)$	Horizontal Pressure	$P_4 = 10594.03\,plf$
$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - Base)$	Resultant Vertical Location	$Y_{P4} = 8.67\,ft$
$P := P_1 + P_2 + P_3 + P_4$	Total Resultant Horizontal Load	$P = 26942.52\,plf$
$Y_P := \frac{\left[(P_1 \cdot Y_{P1}) + (P_2 \cdot Y_{P2}) + (P_3 \cdot Y_{P3}) + (P_4 \cdot Y_{P4})\right]}{P}$	Total Vertical Resultant Location	$Y_P = 12.86\,ft$

Weight of Wall

$W_A := \left(\frac{1}{2}\right)(18\cdot ft)\cdot(8\cdot ft)\cdot\gamma_m$ $\qquad W_A = 7920.00\,plf$

$W_B := (18\cdot ft)\cdot(27\cdot ft)\cdot\gamma_{sat\_fill}$ $\qquad W_B = 60750.00\,plf$

$Weight := W_A + W_B$ $\qquad Weight = 68670.00\,plf$

Moment Arms of Elements (About Toe)

$Arm_A := \frac{2(18\cdot ft)}{3}$ $\qquad Arm_A = 12.00\,ft$

$Arm_B := \frac{(18\cdot ft)}{2}$ $\qquad Arm_B = 9.00\,ft$

Resultant Location of Weight

$$X_W := \frac{W_A\cdot Arm_A + W_B\cdot Arm_B}{Weight}$$

$X_W = 9.35\,ft$ Horizontal distance from toe to resultant

Resultant Vertical Stress

$N := Weight - U$ $\qquad N = 39466.80\,plf$

Sliding Resistance

$\tau_{ult} := N\cdot\tan(\delta_s)$ $\qquad \tau_{ult} = 19249.24\,\frac{lb}{ft}$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $\qquad FS_{sliding\_actual} = 0.71$ $<$ $FS_{sliding\_required} = 1.50$ Fails

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2}\right)}{N} \qquad X_N = 0.82\,ft$$

$$e := \left|X_N - \frac{B}{2}\right| \qquad e = 8.18\,ft$$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B}\right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 13.68\%$ base in compression < $Pct_{comp\_required} = 100.00\%$

Account for Resistance of Passive Soil

Note: Sediment was visually seen from above the water surface to extend up to within a few feet of the surface at the east end corners. Soundings most likely were not capable of capturing this due to debris. Average depth of the canal is -20' LWD (EL. 549.2). Top of rock is approximately 544.4, say 5' of passive sediment can be relied upon. Sediment assumed to be very loose silt.

$$h_{silt} := 5 \cdot ft$$

$$\gamma_{e\_silt\_sat} := 115 \cdot pcf - \gamma_w \qquad \gamma_{e\_silt\_sat} = 52.60\,pcf$$

$$\phi_{silt} := 27 \cdot deg$$

Note: SMF is not used for calculating passive earth pressure coefficients per Reference 3, Para. 3-13.b.

Vertical Stress at Base - Passive Side

Vertical Stress at Base:

$$\sigma_{ybase\_silt} := \gamma_{e\_silt\_sat} \cdot (h_{silt}) \qquad \sigma_{ybase\_silt} = 263.00\,psf$$

Horizontal At Rest Earth Pressure Coefficient:

$$K_p := \left(\tan\left(45 \cdot deg + \frac{\phi_{silt}}{2}\right)\right)^2 \qquad K_p = 2.66$$

(Reference 3, eq. 3-20)

Note: Wall Friction is neglected due to irregular profile.

Latteral Earth Pressure at Base of Structure

$$\sigma_{p\_bot\_silt} := K_p \cdot (\sigma_{ybase\_silt}) \qquad \sigma_{p\_bot\_silt} = 700.35\,psf$$

Horizontal Pressure $P_{silt} := .5 \cdot (\sigma_{p\_bot\_silt}) \cdot (h_{silt})$ $P_{silt} = 1750.88\,ft\,psf$

Resultant Vertical Location $Y_{Psilt} := \frac{(h_{silt})}{3}$ $Y_{Psilt} = 1.67\,ft$

Sliding Factor of Safety $FS_{sliding\_actual} := \frac{\tau_{ult}}{P - \left(\frac{P_{silt}}{2}\right)}$ $FS_{sliding\_actual} = 0.74$

$< FS_{sliding\_required} = 1.50$ Fails

Note: Reference 3, Para. 3-8.b limits passive resistance for stability calculations to 1/2 that of calculated passive resistance.

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2} + P_{silt} \cdot Y_{Psilt}\right)}{N}$$ $X_N = 0.89\,ft$

$$e := \left|X_N - \frac{B}{2}\right|$$ $e = 8.11\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B}\right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 14.91\%$ base in compression $<$ $Pct_{comp\_required} = 100.00\%$

If stabilization methods were implemented, what would be the necessary height of the passive soil?

Note: Assume a stone berm is placed. Stone will be placed on top of silt, silt will not be removed, so silt will act as part of berm. Use parameters of 118 pcf and $\phi = 30$ deg for stone. As both of these parameters are similar to those of the silt, it is conservative to use the parameters for the silt for the design. Assume the slope of the berm is set for 1 on 2 or 26.5 degrees below horizontal. Assume maximum allowable height of berm to be at top of crib (25').

$h_{silt} := 25 \cdot ft$ $\beta_{berm} := -9 \cdot deg$

$\gamma_{e\_silt\_sat} := 115 \cdot pcf - \gamma_w$ $\gamma_{e\_silt\_sat} = 52.60\,pcf$

$\phi_{silt} := 27 \cdot deg$

Note: SMF is not used for calculating passive earth pressure coefficients per Reference 3, Para. 3-13.b.

Vertical Stress at Base - Passive Side

Vertical Stress at Base: $\sigma_{ybase\_silt} := \gamma_{e\_silt\_sat} \cdot (h_{silt})$ $\sigma_{ybase\_silt} = 1315.00\,psf$

Horizontal At Rest Earth Pressure Coefficient:

$$K_p := \frac{(\cos(\phi_{silt}))^2}{\left(1 - \sqrt{\frac{\sin(\phi_{silt}) \cdot \sin(\phi_{silt} + \beta_{berm})}{\cos(\beta_{berm})}}\right)^2}$$

$K_p = 2.04$ (Reference 3, eq. 3-20)

Note: Wall Friction is neglected due to irregular profile.

Latteral Earth Pressure at Base of Structure $\sigma_{p\_bot\_silt} := K_p \cdot (\sigma_{ybase\_silt})$ $\sigma_{p\_bot\_silt} = 2688.72\,psf$

Horizontal Pressure $P_{silt} := .5 \cdot (\sigma_{p\_bot\_silt}) \cdot (h_{silt})$ $P_{silt} = 33609.03\,ft\,psf$

Resultant Vertical Location $Y_{Psilt} := \frac{(h_{silt})}{3}$ $Y_{Psilt} = 8.33\,ft$

Sliding Factor of Safety $FS_{sliding\_actual} := \frac{\tau_{ult}}{P - \left(\frac{P_{silt}}{2}\right)}$ $FS_{sliding\_actual} = 1.90$

$>$ $FS_{sliding\_required} = 1.50$ O.K.

Note: Reference 3, Para. 3-8.b limits passive resistance for stability calculations to 1/2 that of calculated passive resistance.

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2} + \frac{P_{silt}}{2} \cdot Y_{Psilt}\right)}{N}$$ $X_N = 4.37\,ft$

$e := \left|X_N - \frac{B}{2}\right|$ $e = 4.63\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B}\right]$$

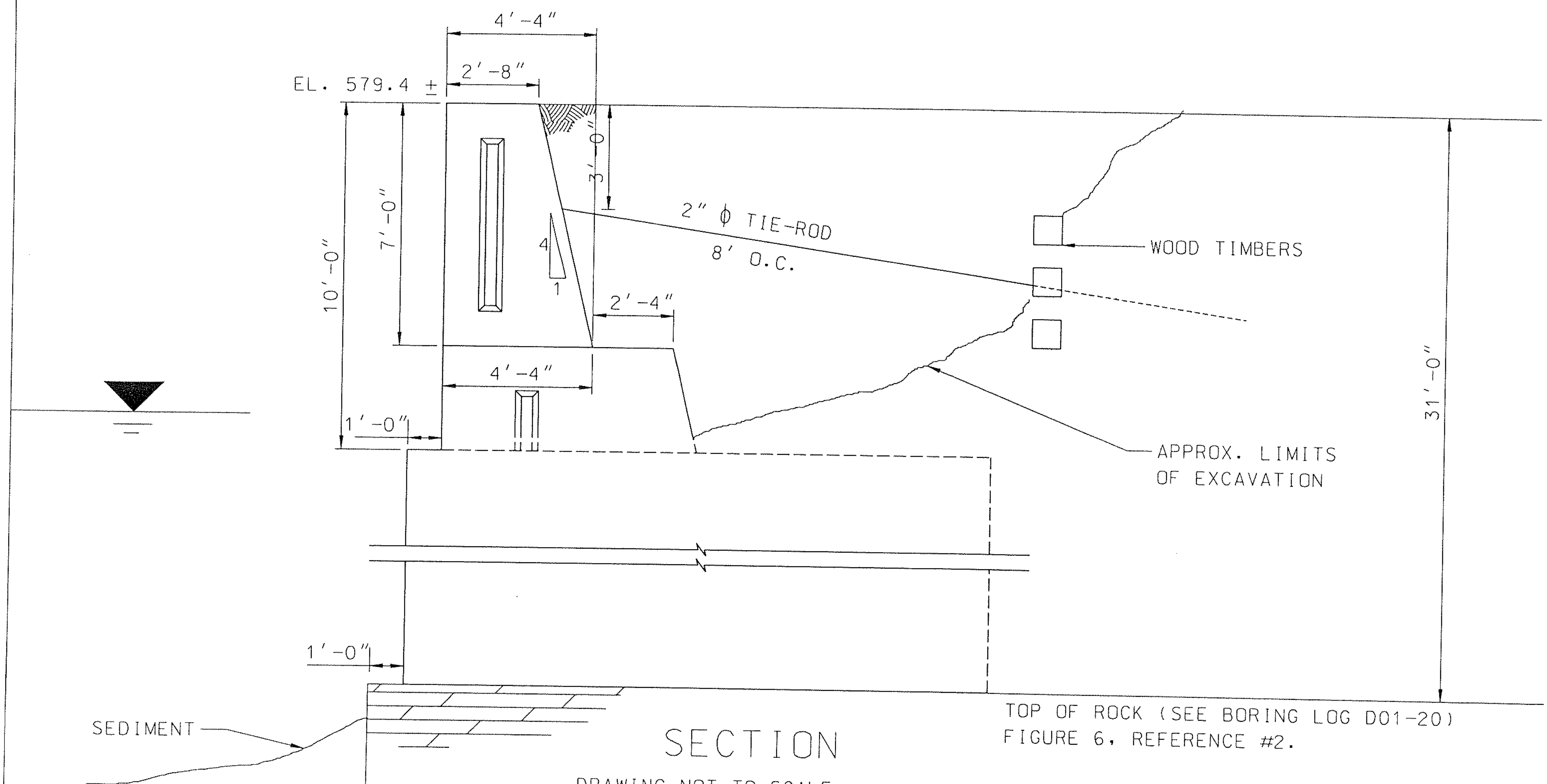
Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 72.82\%$ base in compression $<$ $Pct_{comp\_required} = 100.00\%$ **FAILS**

Note: 73% base in compression is close to the allowable of 75% for existing structures. Based on the conservative assumptions made for the load case and by judgement, consider acceptable. A slope of (-9) deg. is the natural slope that the soil would be stable at under water, see slope stability calculation. No benefit would be derived by using rock to make the angle steeper. Use dredged material.

UNION SHIP CANAL - REACH G

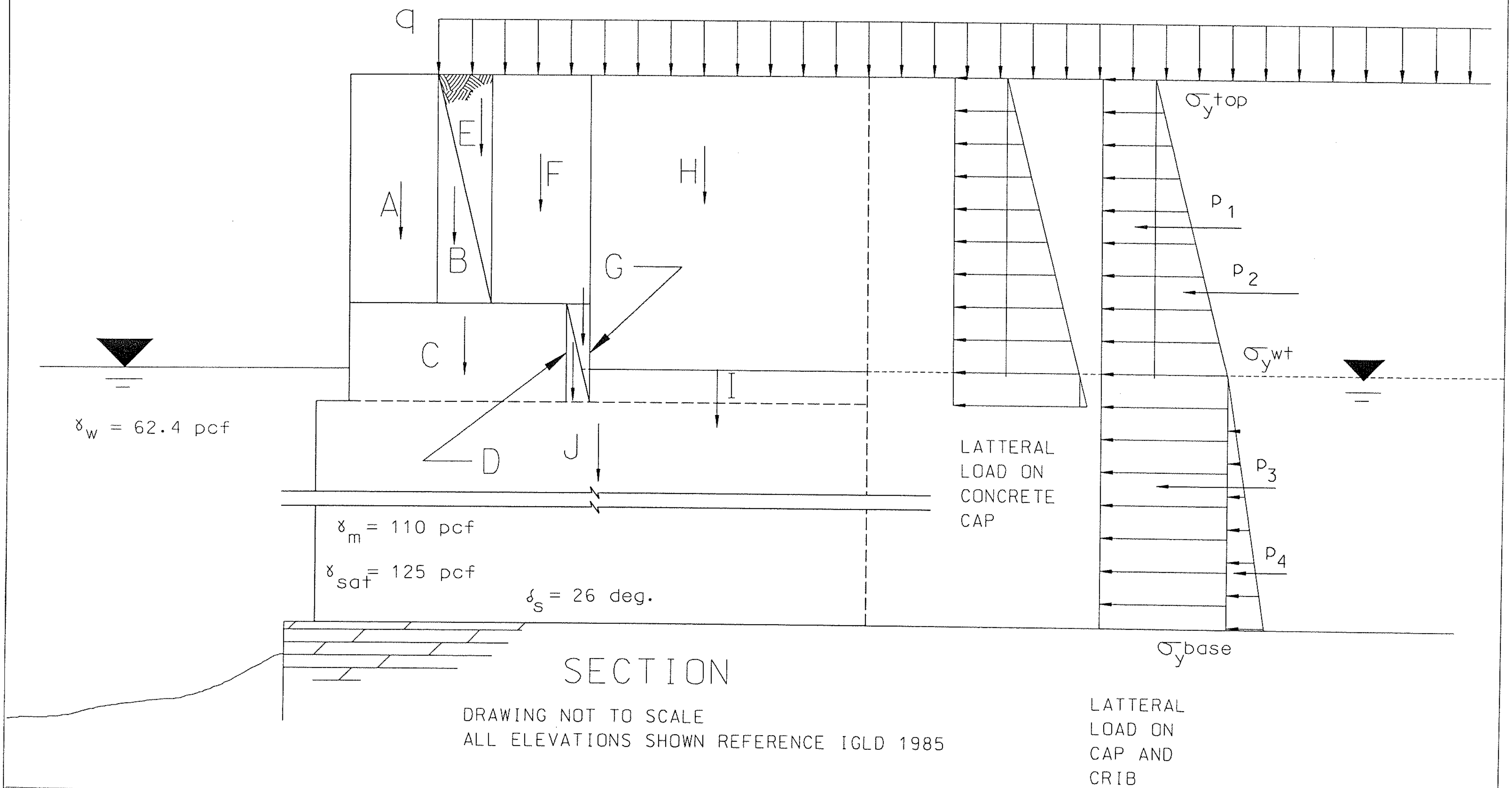
(REPRESENTATIVE OF NORTH WALL STA. 12+00 TO 13+00)



SECTION

DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

UNION SHIP CANAL - REACH G



SECTION

DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

Stability Anal. of Reach G - Top Cap

Material Properties

Water

$\gamma_w := 62.4 \cdot pcf$ Unit Weight of Water

Concrete

$\gamma_c := 145 \cdot pcf$ Unit Weight of Concrete

$f'_c := 6000 \cdot psi$ Compressive Strength of Concrete (Reference 1 - Table 1, Page 3)

Soil

Note: Soil parameters used for stability of the top cap are for the course grained fill. Soil parameters used for stability of the entire crib are taken from the more critical values of the course grained fill and the predominantly clay glacial deposits. See reference #2.

$\gamma_m := 110 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 6)

$\gamma_{sat} := 127 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 6)

$\phi := 31 \cdot deg$ Angle of Internal Friction (Reference 2 - Figure 6)

$\delta_{conc} := 20 \cdot deg$ Angle of Wall Friction (Concrete) (Reference 2 - Figure 6)

$\delta_{crib} := 27 \cdot deg$ Angle of Wall Friction (Cribbing) (Reference 2 - Figure 6)

$\gamma_e := (\gamma_{sat} - \gamma_w)$ Effective Unit Weight of Submerged Fill $\gamma_e = 64.60 pcf$

$\delta_s := 26 \cdot deg$ Shale (fill) Interface Friction Angle (Reference 2 - Figure 6)
(Conservatively assume same for concrete to concrete)

Crib Fill

$\gamma_{m\_fill} := 110 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 6)

$\gamma_{sat\_fill} := 127 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 6)

Design Parameters

$SMF := \frac{2}{3}$ Strength Mobilization Factor (Reference 3, Par. 3-13.b)

$\phi_d := atan(SMF \cdot tan(\phi))$ Developed Shear Strength $\phi_d = 21.83 deg$
(Reference 3, eq. 3-10)

$FS_{sliding\_required} := 1.50$ Minimum Sliding Factor of Safety (Reference 3, Table 4-1)

$Pct_{comp\_required} := 100\%$ Minimum Percent Base in Compression (Reference 3, Table 4-1)

Analysis of Top Cap

Structure Dimensions

$B := 4.33 \cdot ft$ Base Width

Structure Elevations

$Top := 7 \cdot ft$ Top of Structure

$Base := 0 \cdot ft$ Base of Structure

$W := 0 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf$ Surcharge (Snow)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot B$ $U = 0.00\,plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 100.00\,psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 870.00\,psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 870.00\,psf$

Latteral Earth Pressure Consider Wall Friction due to straight back and uniform material

$\beta := 0 \cdot deg$ Angle of Top of Fill from Horizontal

$\theta := \operatorname{atan}\left(\frac{7 \cdot ft}{1.667 \cdot ft}\right)$ $\theta = 76.60 deg$ Angle of Wall Face from Horizontal

Horizontal At Rest Earth Pressure Coefficient:

$$K_O := \frac{(\sin(\theta + \phi_d))^2 \cdot \cos(\delta_{conc})}{\sin(\theta) \cdot \sin(\theta - \delta_{conc}) \cdot \left(1 + \sqrt{\frac{\sin(\phi_d + \delta_{conc}) \cdot \sin(\phi_d - \beta)}{\sin(\theta - \delta_{conc}) \cdot \sin(\theta + \beta)}}\right)^2}$$

$K_O = 0.47$ (Reference 3, eq. 3-12)

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$ Latteral Earth Pressure at Top of Structure $\sigma_{a\_top} = 46.97 psf$

$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$ Latteral Earth Pressure at Water Table $\sigma_{a\_wt} = 408.60 psf$

$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$ Latteral Earth Pressure at Base of Structure $\sigma_{a\_bot} = 408.60 psf$

$P_1 := (\sigma_{a\_top}) \cdot (Top - W)$ Horizontal Pressure $P_1 = 328.76 ft psf$

$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (Top - W)\right]$ Resultant Vertical Location $Y_{P1} = 3.50 ft$

$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (Top - W)$ Horizontal Pressure $P_2 = 1265.72 plf$

$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (Top - W)\right]$ Resultant Vertical Location $Y_{P2} = 2.33 ft$

$P_3 := \sigma_{a\_wt} \cdot (W - Base)$ Horizontal Pressure $P_3 = 0.00 plf$

$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - Base)\right]$ Resultant Vertical Location $Y_{P3} = 0.00 ft$

$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - Base)$ Horizontal Pressure $P_4 = 0.00 plf$

$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - Base)$ Resultant Vertical Location $Y_{P4} = 0.00 ft$

$P := P_1 + P_2 + P_3 + P_4$ Total Resultant Horizontal Load $P = 1594.48 plf$

$$Y_P := \frac{\left[\left(P_1 \cdot Y_{P1}\right) + \left(P_2 \cdot Y_{P2}\right) + \left(P_3 \cdot Y_{P3}\right) + \left(P_4 \cdot Y_{P4}\right)\right]}{P} \qquad Y_P = 2.57\,\text{ft}$$

Total Vertical Resultant Location

Weight of Wall

$$W_A := (7\cdot \text{ft})\cdot(6\cdot \text{ft})\cdot\gamma_c \qquad W_A = 6090.00\,\text{plf}$$

$$W_B := \left(\frac{1}{2}\right)\cdot(2\cdot \text{ft})\cdot(6\cdot \text{ft})\cdot\gamma_c \qquad W_B = 870.00\,\text{plf}$$

$$W_C := \left(0\cdot \text{ft}^2\right)\cdot\gamma_c \qquad W_C = 0.00\,\text{plf}$$

$$W_D := \left(0\cdot \text{ft}^2\right)\cdot\gamma_c \qquad W_D = 0.00\,\text{plf}$$

$$W_E := \left(\frac{1}{2}\right)\cdot(1.667\cdot \text{ft})\cdot(7\cdot \text{ft})\cdot\gamma_m \qquad W_E = 641.80\,\text{plf}$$

$$W_F := \left(0\cdot \text{ft}^2\right)\cdot\gamma_m \qquad W_F = 0.00\,\text{plf}$$

$$\text{Weight} := W_A + W_B + W_C + W_D + W_E + W_F \qquad \text{Weight} = 7601.80\,\text{plf}$$

Moment Arms of Elements (About Toe)

$$\text{Arm}_A := \frac{(2.667\cdot \text{ft})}{2} \qquad \text{Arm}_A = 1.33\,\text{ft}$$

$$\text{Arm}_B := (2.667\cdot \text{ft}) + \frac{(1.667\cdot \text{ft})}{3} \qquad \text{Arm}_B = 3.22\,\text{ft}$$

$$\text{Arm}_C := 0\cdot \text{ft} \qquad \text{Arm}_C = 0.00\,\text{ft}$$

$$\text{Arm}_D := 0\cdot \text{ft} \qquad \text{Arm}_D = 0.00\,\text{ft}$$

$$\text{Arm}_E := (7\cdot \text{ft}) + (2\cdot \text{ft})\cdot\frac{2}{3} \qquad \text{Arm}_E = 8.33\,\text{ft}$$

$$\text{Arm}_F := 0\cdot \text{ft} \qquad \text{Arm}_F = 0.00\,\text{ft}$$

Resultant Location of Weight

$$X_W := \frac{W_A\cdot \text{Arm}_A + W_B\cdot \text{Arm}_B + W_C\cdot \text{Arm}_C + W_D\cdot \text{Arm}_D + W_E\cdot \text{Arm}_E + W_F\cdot \text{Arm}_F}{\text{Weight}}$$

$X_W = 2.14\,\text{ft}$ Horizontal distance from toe to center of mass

Resultant Vertical Stress

$N := \text{Weight} - U$ $N = 7601.80\,\text{plf}$

Sliding Resistance

$\tau_{ult} := N \cdot \tan(\delta_s)$ $\tau_{ult} = 3707.64\,\dfrac{\text{lb}}{\text{ft}}$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$FS_{sliding\_actual} := \dfrac{\tau_{ult}}{P}$ $FS_{sliding\_actual} = 2.33$ > $FS_{sliding\_required} = 1.50$ O.K.

Resultant location (from center of base)

$$X_N := \frac{\left(\text{Weight} \cdot X_W - P \cdot Y_P - U \cdot \dfrac{B}{2}\right)}{N}$$ $X_N = 1.60\,\text{ft}$

$$e := \left| X_N - \frac{B}{2} \right|$$ $e = 0.56\,\text{ft}$

$$Pct_{comp\_actual} := \text{if}\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\dfrac{B}{2} - e\right)}{B}\right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 100.00\,\%$ base in compression > $Pct_{comp\_required} = 100.00\,\%$

Stability Anal. of Reach G - Middle & Top Cap

Material Properties

Water	$\gamma_w := 62.4 \cdot pcf$	Unit Weight of Water	
Concrete	$\gamma_c := 145 \cdot pcf$	Unit Weight of Concrete	
	$f'_c := 6000 \cdot psi$	Compressive Strength of Concrete	(Reference 1 - Table 1, Page 3)
Soil	$\gamma_m := 110 \cdot pcf$	Moist Unit Weight	(Reference 2 - Figure 6)
	$\gamma_{sat} := 127 \cdot pcf$	Saturated Unit Weight	(Reference 2 - Figure 6)
	$\phi := 31 \cdot deg$	Angle of Internal Friction	(Reference 2 - Figure 6)
	$\delta_{conc} := 20 \cdot deg$	Angle of Wall Friction (Concrete)	(Reference 2 - Figure 6)
	$\delta_{crib} := 27 \cdot deg$	Angle of Wall Friction (Cribbing)	(Reference 2 - Figure 6)
	$\gamma_e := (\gamma_{sat} - \gamma_w)$	Effective Unit Weight of Submerged Fill	$\gamma_e = 64.60 pcf$
	$\delta_s := 26 \cdot deg$	Shale (fill) Interface Friction Angle	(Reference 2 - Figure 6)
Crib Fill	$\gamma_{m\_fill} := 110 \cdot pcf$	Moist Unit Weight	(Reference 2 - Figure 6)
	$\gamma_{sat\_fill} := 127 \cdot pcf$	Saturated Unit Weight	(Reference 2 - Figure 6)

Note: Soil parameters used for stability of the top cap are for the course grained fill. Soil parameters used for stability of the entire crib are taken from the more critical values of the course grained fill and the predominantly clay glacial deposits. See reference #2.

Design Parameters

$SMF := \frac{2}{3}$	Strength Mobilization Factor	(Reference 3, Par. 3-13.b)
$\phi_d := atan(SMF \cdot tan(\phi))$	Developed Shear Strength	$\phi_d = 21.83 deg$ (Reference 3, eq. 3-10)
$FS_{sliding\_required} := 1.50$	Minimum Sliding Factor of Safety	(Reference 3, Table 4-1)
$Pct_{comp\_required} := 100\%$	Minimum Percent Base in Compression	(Reference 3, Table 4-1)

Analysis of Top Cap (Ignore stability contributed by tie-rods)

Structure Dimensions

$B := 7.42 \cdot ft$ Base Width

Structure Elevations

$Top := 10 \cdot ft$ Top of Structure

$Base := 0 \cdot ft$ Base of Structure

$W := 1 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf$ Surcharge (Snow)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot B$ $U = 463.01\, plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 100.00\, psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 1090.00\, psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 1154.60\, psf$

Latteral Earth Pressure

Horizontal At Rest Earth Pressure Coefficient: $K_O := \left(\tan\left(45 \cdot deg - \frac{\phi d}{2}\right)\right)^2$ $K_O = 0.46$ (Reference 3, eq. 3-15)

Note: Wall Friction is neglected due to presence of the heel.

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$	Latteral Earth Pressure at Top of Structure	$\sigma_{a\_top} = 45.79\,psf$
$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$	Latteral Earth Pressure at Water Table	$\sigma_{a\_wt} = 499.10\,psf$
$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$	Latteral Earth Pressure at Base of Structure	$\sigma_{a\_bot} = 528.67\,psf$
$P_1 := (\sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_1 = 412.10\,ft\,psf$
$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P1} = 5.50\,ft$
$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_2 = 2039.88\,plf$
$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P2} = 4.00\,ft$
$P_3 := \sigma_{a\_wt} \cdot (W - Base)$	Horizontal Pressure	$P_3 = 499.10\,plf$
$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - Base)\right]$	Resultant Vertical Location	$Y_{P3} = 0.50\,ft$
$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - Base)$	Horizontal Pressure	$P_4 = 14.79\,plf$
$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - Base)$	Resultant Vertical Location	$Y_{P4} = 0.33\,ft$
$P := P_1 + P_2 + P_3 + P_4$	Total Resultant Horizontal Load	$P = 2965.86\,plf$
$Y_P := \frac{\left[(P_1 \cdot Y_{P1}) + (P_2 \cdot Y_{P2}) + (P_3 \cdot Y_{P3}) + (P_4 \cdot Y_{P4})\right]}{P}$	Total Vertical Resultant Location	$Y_P = 3.60\,ft$

Weight of Wall

$W_A := (7\cdot ft)\cdot(2.667\cdot ft)\cdot\gamma_c$ $\quad W_A = 2707.00\,plf$

$W_B := \left(\frac{1}{2}\right)\cdot(1.667\cdot ft)\cdot(7\cdot ft)\cdot\gamma_c$ $\quad W_B = 846.00\,plf$

$W_C := (3\cdot ft)\cdot(6.667\cdot ft)\cdot\gamma_c$ $\quad W_C = 2900.14\,plf$

$W_D := \left(\frac{1}{2}\right)\cdot(0.75\cdot ft)\cdot(3\cdot ft)\cdot\gamma_c$ $\quad W_D = 163.13\,plf$

$W_E := \left(\frac{1}{2}\right)\cdot(1.667\cdot ft)\cdot(7\cdot ft)\cdot\gamma_m$ $\quad W_E = 641.80\,plf$

$W_F := (3.08\cdot ft)\cdot(7\cdot ft)\cdot\gamma_m$ $\quad W_F = 2371.60\,plf$

$W_G := \left(\frac{0.75\cdot ft + 0.25\cdot ft}{2}\right)\cdot(2\cdot ft)\cdot\gamma_m + \left(\frac{1}{2}\right)\cdot(0.25\cdot ft)\cdot(1\cdot ft)\cdot(\gamma_{sat} - \gamma_m)$ $\quad W_G = 112.13\,plf$

$Weight := W_A + W_B + W_C + W_D + W_E + W_F + W_G$ $\quad Weight = 9741.80\,plf$

Moment Arms of Elements (About Toe)

$Arm_A := \frac{(2.667\cdot ft)}{2}$ $\quad Arm_A = 1.33\,ft$

$Arm_B := (2.667\cdot ft) + \frac{(1.667\cdot ft)}{3}$ $\quad Arm_B = 3.22\,ft$

$Arm_C := \frac{(4.333\cdot ft)}{2}$ $\quad Arm_C = 2.17\,ft$

$Arm_D := \left(4.333\cdot ft + \frac{0.75}{3}\,ft\right)$ $\quad Arm_D = 4.58\,ft$

$Arm_E := (2.667\cdot ft) + (1.667\cdot ft)\cdot\frac{2}{3}$ $\quad Arm_E = 3.78\,ft$

$Arm_F := (4.333\cdot ft) + \frac{(3.08\cdot ft)}{2}$ $\quad Arm_F = 5.87\,ft$

$Arm_G := (6.667\cdot ft) + (0.75\cdot ft)\cdot\frac{2}{3}$ $\quad Arm_G = 7.17\,ft$

Resultant Location of Weight

$$X_W := \frac{W_A \cdot Arm_A + W_B \cdot Arm_B + W_C \cdot Arm_C + W_D \cdot Arm_D + W_E \cdot Arm_E + W_F \cdot Arm_F + W_G \cdot Arm_G}{Weight}$$

$X_W = 3.11\,ft$ Horizontal distance from toe to center of mass

Resultant Vertical Stress

$N := Weight - U$ $N = 9223.79\,plf$

Sliding Resistance

$\tau_{ult} := N \cdot \tan(\delta_s)$ $\tau_{ult} = 4498.74\,\frac{lb}{ft}$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $FS_{sliding\_actual} = 1.52$ > $FS_{sliding\_required} = 1.50$ O.K.

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2}\right)}{N}$$ $X_N = 1.92\,ft$

$$e := \left|X_N - \frac{B}{2}\right|$$ $e = 1.79\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B}\right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 77.72\,\%$ base in compression < $Pct_{comp\_required} = 100.00\,\%$ **FAILS**

Analyse Assuming Tie-Rods are Effective

In the above analyses, the tie-rods were assumed to be ineffective as it was not possible to determine their condition. For comparison, assume that the tie-rods are effective and re-analyse. Additionally, a resisting shear force can be theorized at the interface between the bottom of the cap and the top of the shale crib fill. This shear force would be limited to the lower value of: (1) the excess capacity of the anchors, (2) the sliding resistance of the concrete cap on the shale fill and (3) the capacity of the anchorage. As no information is available on the anchorage, it will be assumed that capacity of the anchorage does not control.

Determine Resisting Force Capacity of Tie-Rods

$A_{tr} := \frac{1}{4} \cdot \pi \cdot (1.5 \cdot in)^2$ Area of Tie-Rods $A_{tr} = 1.77\,in^2$

$T_{allow} := A_{tr} \cdot [(0.6) \cdot (36 \cdot ksi)]$ Allowable Tension $T_{allow} = 38170.35\,lb$

$Cap_{lat} := \frac{T_{allow}}{8 \cdot ft}$ Latteral Capacity on Per Foot Basis $Cap_{lat} = 4771.29\,plf$

Note: Assume A36, very conservative assumption. Strength most likely much greater.

Determine Required Load in Tie-Rods for Sliding Stability

$F_{TR\_sliding} := 0 \cdot plf$ Assumed load in tie-rod, itterated by trial and error, to meet sliding F.S.

$\tau_{ult} := N \cdot \tan(\delta_s) + F_{TR\_sliding}$ $\tau_{ult} = 4525.57\,\frac{lb}{ft}$ (Reference 3, eq. 4-5)

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $FS_{sliding\_actual} = 1.53$ = $FS_{sliding\_required} = 1.50$

Determine Required Load in Tie-Rods for Overturning Stability

$F_{TR\_ovt} := 207 \cdot plf$ Assumed load in tie-rod, itterated by trial and error, to meet overturning F.S.

$$X_N := \frac{\left[Weight \cdot X_W + F_{TR\_ovt}(24 \cdot ft) - P \cdot Y_P - U \cdot \frac{B}{2}\right]}{N}$$ $X_N = 2.49\,ft$

$e := \left|X_N - \frac{B}{2}\right|$ $e = 1.22\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B}\right]$$ Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 100.00\%$ base in compression = $Pct_{comp\_required} = 100.00\%$

Controlling Tie-Rod Load

$F_{TR} := \max(F_{TR\_sliding}, F_{TR\_ovt})$ $F_{TR} = 207.00 \frac{lb}{ft}$ < $Cap_{lat} = 4771.29 \frac{lb}{ft}$ O.K.

Check F.S. Against Bulging (Reference 4, Par. 5.7.2)

$P'_a := 1594.48 \cdot plf$ Resultant Horizontal Load on Upper Portion of Concrete Cap, from Calculations in **G Top.mcd**

$F_g := F_{TR}$ Horizontal Load in Tie-Rods as Required

$V_h := 3707.64 \cdot plf$ Sliding Resistance of Concrete Cap on Concrete, from Calculations in **G Top.mcd**

$FS_{bulging} := \frac{V_h}{|P'_a - F_g|}$ Factor of Safety Against Bulging $FS_{bulging} = 2.67$ > 1.5 O.K.

Stability Anal. of Reach G - Crib

Material Properties

Water

$\gamma_w := 62.4 \cdot pcf$ Unit Weight of Water

Concrete

$\gamma_c := 145 \cdot pcf$ Unit Weight of Concrete

$f'_c := 6000 \cdot psi$ Compressive Strength of Concrete (Reference 1 - Table 1, Page 3)

Soil

Note: Soil parameters used for stability of the top cap are for the course grained fill. Soil parameters used for stability of the entire crib are taken from the more critical values of the course grained fill and the predominantly clay glacial deposits. See reference #2.

$\gamma_m := 110 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 6)

$\gamma_{sat} := 127 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 6)

$\phi := 29 \cdot deg$ Angle of Internal Friction (Reference 2 - Figure 6)

$\delta_{conc} := 20 \cdot deg$ Angle of Wall Friction (Concrete) (Reference 2 - Figure 6)

$\delta_{crib} := 27 \cdot deg$ Angle of Wall Friction (Cribbing) (Reference 2 - Figure 6)

$\gamma_e := (\gamma_{sat} - \gamma_w)$ Effective Unit Weight of Submerged Fill $\gamma_e = 64.60 pcf$

$\delta_s := 26 \cdot deg$ Shale (fill) Interface Friction Angle (Reference 2 - Figure 6)

Crib Fill

$\gamma_{m\_fill} := 110 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 6)

$\gamma_{sat\_fill} := 127 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 6)

Design Parameters

$SMF := \frac{2}{3}$ Strength Mobilization Factor (Reference 3, Par. 3-13.b)

$\phi_d := atan(SMF \cdot tan(\phi))$ Developed Shear Strength $\phi_d = 20.28 deg$

(Reference 3, eq. 3-10)

$FS_{sliding\_required} := 1.50$ Minimum Sliding Factor of Safety (Reference 3, Table 4-1)

$Pct_{comp\_required} := 100\%$ Minimum Percent Base in Compression (Reference 3, Table 4-1)

Analysis of Crib (Ignore stability contributed by tie-rods)

Structure Dimensions

$B := 18 \cdot ft$ Base Width

Structure Elevations

$Top := 31 \cdot ft$ Top of Structure

$Base := 0 \cdot ft$ Base of Structure

$W := 22 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf$ Surcharge (Snow)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot B$ $U = 24710.40\,plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 100.00\,psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 1090.00\,psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 2511.20\,psf$

Latteral Earth Pressure

Horizontal At Rest Earth Pressure Coefficient: $K_O := \left(\tan\left(45 \cdot deg - \frac{\phi_d}{2}\right)\right)^2$ $K_O = 0.49$ (Reference 3, eq. 3-15)

Note: Wall Friction is neglected due to presence of the heel.

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$	Latteral Earth Pressure at Top of Structure	$\sigma_{a\_top} = 48.52\,psf$
$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$	Latteral Earth Pressure at Water Table	$\sigma_{a\_wt} = 528.86\,psf$
$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$	Latteral Earth Pressure at Base of Structure	$\sigma_{a\_bot} = 1218.41\,psf$
$P_1 := (\sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_1 = 436.67\,ft\,psf$
$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P1} = 26.50\,ft$
$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_2 = 2161.52\,plf$
$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P2} = 25.00\,ft$
$P_3 := \sigma_{a\_wt} \cdot (W - Base)$	Horizontal Pressure	$P_3 = 11634.86\,plf$
$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - Base)\right]$	Resultant Vertical Location	$Y_{P3} = 11.00\,ft$
$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - Base)$	Horizontal Pressure	$P_4 = 7585.08\,plf$
$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - Base)$	Resultant Vertical Location	$Y_{P4} = 7.33\,ft$
$P := P_1 + P_2 + P_3 + P_4$	Total Resultant Horizontal Load	$P = 21818.13\,plf$
$Y_P := \frac{\left[(P_1 \cdot Y_{P1}) + (P_2 \cdot Y_{P2}) + (P_3 \cdot Y_{P3}) + (P_4 \cdot Y_{P4})\right]}{P}$	Total Vertical Resultant Location	$Y_P = 11.42\,ft$

Weight of Wall

$W_A := (7\cdot ft)\cdot(2.667\cdot ft)\cdot\gamma_c$ $\qquad W_A = 2707.00\,plf$

$W_B := \left(\frac{1}{2}\right)\cdot(1.667\cdot ft)\cdot(7\cdot ft)\cdot\gamma_c$ $\qquad W_B = 846.00\,plf$

$W_C := (3\cdot ft)\cdot(6.667\cdot ft)\cdot\gamma_c$ $\qquad W_C = 2900.14\,plf$

$W_D := \left(\frac{1}{2}\right)\cdot(0.75\cdot ft)\cdot(3\cdot ft)\cdot\gamma_c$ $\qquad W_D = 163.13\,plf$

$W_E := \left(\frac{1}{2}\right)\cdot(1.667\cdot ft)\cdot(7\cdot ft)\cdot\gamma_m$ $\qquad W_E = 641.80\,plf$

$W_F := (3.08\cdot ft)\cdot(7\cdot ft)\cdot\gamma_m$ $\qquad W_F = 2371.60\,plf$

$W_G := \left(\frac{0.75\cdot ft + 0.25\cdot ft}{2}\right)\cdot(2\cdot ft)\cdot\gamma_m + \left(\frac{1}{2}\right)\cdot(0.25\cdot ft)\cdot(1\cdot ft)\cdot(\gamma_{sat} - \gamma_m)$ $\qquad W_G = 112.13\,plf$

$W_H := (9\cdot ft)\cdot(9.58\cdot ft)\cdot\gamma_m$ $\qquad W_H = 9484.20\,plf$

$W_I := (1\cdot ft)\cdot(9.58\cdot ft)\cdot\gamma_{sat}$ $\qquad W_I = 1216.66\,plf$

$W_J := B\cdot(21\cdot ft)\cdot\gamma_{sat\_fill}$ $\qquad W_J = 48006.00\,plf$

$Weight := W_A + W_B + W_C + W_D + W_E + W_F + W_G + W_H + W_I + W_J$ $\qquad Weight = 68448.66\,plf$

Moment Arms of Elements (About Toe)

$Arm_A := \frac{(2.667\cdot ft)}{2} + (1\cdot ft)$ $\qquad Arm_A = 2.33\,ft$

$Arm_B := (2.667\cdot ft) + \frac{(1.667\cdot ft)}{3} + (1\cdot ft)$ $\qquad Arm_B = 4.22\,ft$

$Arm_C := \frac{(4.333\cdot ft)}{2} + (1\cdot ft)$ $\qquad Arm_C = 3.17\,ft$

$Arm_D := \left(4.333\cdot ft + \frac{0.75}{3}\cdot ft\right) + (1\cdot ft)$ $\qquad Arm_D = 5.58\,ft$

$Arm_E := (2.667\cdot ft) + (1.667\cdot ft)\cdot\frac{2}{3} + (1\cdot ft)$ $\qquad Arm_E = 4.78\,ft$

$Arm_F := (4.333\cdot ft) + \frac{(3.08\cdot ft)}{2} + (1\cdot ft)$ $\qquad Arm_F = 6.87\,ft$

$$Arm_G := (6.667\cdot ft) + (0.75\cdot ft)\cdot\frac{2}{3} + (1\cdot ft) \qquad Arm_G = 8.17\,ft$$

$$Arm_H := 6.667\cdot ft + 0.75\cdot ft + \frac{9.58\cdot ft}{2} + (1\cdot ft) \qquad Arm_H = 13.21\,ft$$

$$Arm_I := 6.667\cdot ft + 0.75\cdot ft + \frac{9.58\cdot ft}{2} + (1\cdot ft) \qquad Arm_I = 13.21\,ft$$

$$Arm_J := \frac{B}{2} \qquad Arm_J = 9.00\,ft$$

Resultant Location of Weight

$$X_W := \frac{W_A\cdot Arm_A + W_B\cdot Arm_B + W_C\cdot Arm_C + W_D\cdot Arm_D + W_E\cdot Arm_E + W_F\cdot Arm_F + W_G\cdot Arm_G + W_H\cdot Arm_H + W_I\cdot Arm_I + W_J\cdot Arm_J}{Weight}$$

$X_W = 8.97\,ft$ Horizontal distance from toe to center of mass

Resultant Vertical Stress

$$N := Weight - U \qquad N = 43683.26\,plf$$

Sliding Resistance

$$\tau_{ult} := N\cdot\tan(\delta_s) \qquad \tau_{ult} = 21305.75\,plf$$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$$FS_{sliding\_actual} := \frac{\tau_{ult}}{P} \qquad FS_{sliding\_actual} = 0.98 \quad < \quad FS_{sliding\_required} = 1.50$$ Fails

Resultant location (from center of base)

$$X_N := \frac{\left(Weight\cdot X_W - P\cdot Y_P - U\cdot\frac{B}{2}\right)}{N} \qquad X_N = 3.24\,ft$$

$$e := \left|X_N - \frac{B}{2}\right| \qquad e = 5.76\,ft$$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3\cdot\frac{\left(\frac{B}{2} - e\right)}{B}\right]$$ Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 54.02\,\%$ base in compression $<$ $Pct_{comp\_required} = 100.00\,\%$ Fails

Analyse Assuming Tie-Rods are Effective

In the above analyses, the tie-rods were assumed to be ineffective as it was not possible to determine their condition. For comparison, assume that the tie-rods are effective and re-analyse. Additionally, a resisting shear force can be theorized at the interface between the bottom of the cap and the top of the shale crib fill. This shear force would be limited to the lower value of: (1) the excess capacity of the anchors, (2) the sliding resistance of the concrete cap on the shale fill and (3) the capacity of the anchorage. As no information is available on the anchorage, it will be assumed that capacity of the anchorage does not control.

Determine Resisting Force Capacity of Tie-Rods

$A_{tr} := \frac{1}{4} \cdot \pi \cdot (1.5 \cdot in)^2$ Area of Tie-Rods $A_{tr} = 1.77\,in^2$

$T_{allow} := A_{tr} [(0.6) \cdot (36 \cdot ksi)]$ Allowable Tension $T_{allow} = 38170.35\,lb$

$Cap_{lat} := \frac{T_{allow}}{8 \cdot ft}$ Latteral Capacity on Per Foot Basis $Cap_{lat} = 4771.29\,plf$

Note: Assume A36, very conservative assumption. Strength most likely much greater.

Determine Required Load in Tie-Rods for Sliding Stability

$F_{TR\_sliding} := 4771.29 \cdot plf$ Assumed load in tie-rod, iterated by trial and error, to meet sliding F.S., limited to above calculated capacity of tie-rods.

$\tau_{ult} := N \cdot \tan(\delta_s) + F_{TR\_sliding}$ $\tau_{ult} = 26103.86\,plf$ (Reference 3, eq. 4-5)

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $FS_{sliding\_actual} = 1.20$ < $FS_{sliding\_required} = 1.50$ FAILS

Determine Required Load in Tie-Rods for Overturning Stability

$F_{TR\_ovt} := 4771.29 \cdot plf$ Assumed load in tie-rod, itterated by trial and error, to meet overturning F.S., limited to above calculated capacity of tie-rods.

$$X_N := \frac{\left[Weight \cdot X_W + F_{TR\_ovt}(24 \cdot ft) - P \cdot Y_P - U \cdot \frac{B}{2}\right]}{N}$$ $X_N = 5.87\,ft$

$e := \left| X_N - \frac{B}{2} \right|$ $e = 3.13\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B}\right]$$ Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 97.76\,\%$ base in compression < $Pct_{comp\_required} = 100.00\,\%$

Controlling Tie-Rod Load

$F_{TR} := \max(F_{TR\_sliding}, F_{TR\_ovt})$ $F_{TR} = 4771.29\,\text{plf}$ < $Cap_{lat} = 4771.29\,\text{plf}$ O.K.

Check F.S. Against Bulging (Reference 4, Par. 5.7.2)

Top $P'_a := 1594.48 \cdot \text{plf}$ Resultant Horizontal Load on Upper Portion of Concrete Cap, from Calculations in "G Top"

$F_g := F_{TR}$ Horizontal Load in Tie-Rods as Required

$V_h := 3707.64 \cdot \text{plf}$ Sliding Resistance of Concrete Cap on Concrete, from Calculations in "G Top"

$FS_{bulging} := \frac{V_h}{|P'_a - F_g|}$ Factor of Safety Against Bulging $FS_{bulging} = 1.17$ < 1.5 Fails

Mid $P'_a := 2965.86 \cdot \text{plf}$ Resultant Horizontal Load on Upper Portion of Concrete Cap, from Calculations in "G top&mid"

$F_g := F_{TR}$ Horizontal Load in Tie-Rods as Required

$V_h := 4532.27 \cdot \text{plf}$ Sliding Resistance of Concrete Cap on fill, from Calculations in "G top&mid"

$FS_{bulging} := \frac{V_h}{|P'_a - F_g|}$ Factor of Safety Against Bulging $FS_{bulging} = 2.51$ > 1.5 O.K.

Note: There is insufficient capacity in the tie-rods to bring the structure up to requirements for either new or existing construction. The tie-rod system should not be relied upon for stability. Additionally, if the tie-rods are relied upon for stability, there is insufficient factor of safety against bulging at the plane between the top and mid parts of the concrete cap.

If stabilization methods were implemented, what would be the necessary height of the passive soil?

Note: Assume a stone berm is placed. Stone will be placed on top of silt, silt will not be removed, so silt will act as part of berm. Use parameters of 118 pcf and $\phi = 30$ deg for stone. As both of these parameters are similar to those of the silt, it is conservative to use the parameters for the silt for the design. Assume the slope of the berm is set for 1 on 2 or 26.5 degrees below horizontal.

$h_{silt} := 21 \cdot ft$

$\beta_{berm} := -20 \cdot deg$

$\gamma_{e\_silt\_sat} := 115 \cdot pcf - \gamma_w$

$\gamma_{e\_silt\_sat} = 52.60 pcf$

$\phi_{silt} := 27 \cdot deg$

Note: SMF is not used for calculating passive earth pressure coefficients per Reference 3, Para. 3-13.b.

Vertical Stress at Base - Passive Side

Vertical Stress at Base:

$\sigma_{ybase\_silt} := \gamma_{e\_silt\_sat} \left(h_{silt}\right)$

$\sigma_{ybase\_silt} = 1104.60 \, psf$

Horizontal At Rest Earth Pressure Coefficient:

$$K_p := \frac{\left(\cos\left(\phi_{silt}\right)\right)^2}{\left(1 - \sqrt{\frac{\sin\left(\phi_{silt}\right) \cdot \sin\left(\phi_{silt} + \beta_{berm}\right)}{\cos\left(\beta_{berm}\right)}}\right)^2}$$

$K_p = 1.38$ (Reference 3, eq. 3-20)

Note: Wall Friction is neglected due to irregular profile.

Latteral Earth Pressure at Base of Structure

$\sigma_{p\_bot\_silt} := K_p \cdot \left(\sigma_{ybase\_silt}\right)$

$\sigma_{p\_bot\_silt} = 1528.87 \, psf$

Horizontal Pressure

$P_{silt} := .5 \cdot \left(\sigma_{p\_bot\_silt}\right) \cdot \left(h_{silt}\right)$

$P_{silt} = 16053.18 \, ft \, psf$

Resultant Vertical Location

$Y_{Psilt} := \frac{\left(h_{silt}\right)}{3}$

$Y_{Psilt} = 7.00 \, ft$

Sliding Factor of Safety

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P - \left(\frac{P_{silt}}{2}\right)}$

$FS_{sliding\_actual} = 1.89$

$> FS_{sliding\_required} = 1.50$ O.K.

Note: Reference 3, Para. 3-8.b limits passive resistance for stability calculations to 1/2 that of calculated passive resistance.

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2} + \frac{P_{silt}}{2} \cdot Y_{Psilt}\right)}{N} \qquad X_N = 4.53\,ft$$

$$e := \left|X_N - \frac{B}{2}\right| \qquad e = 4.47\,ft$$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B}\right]$$

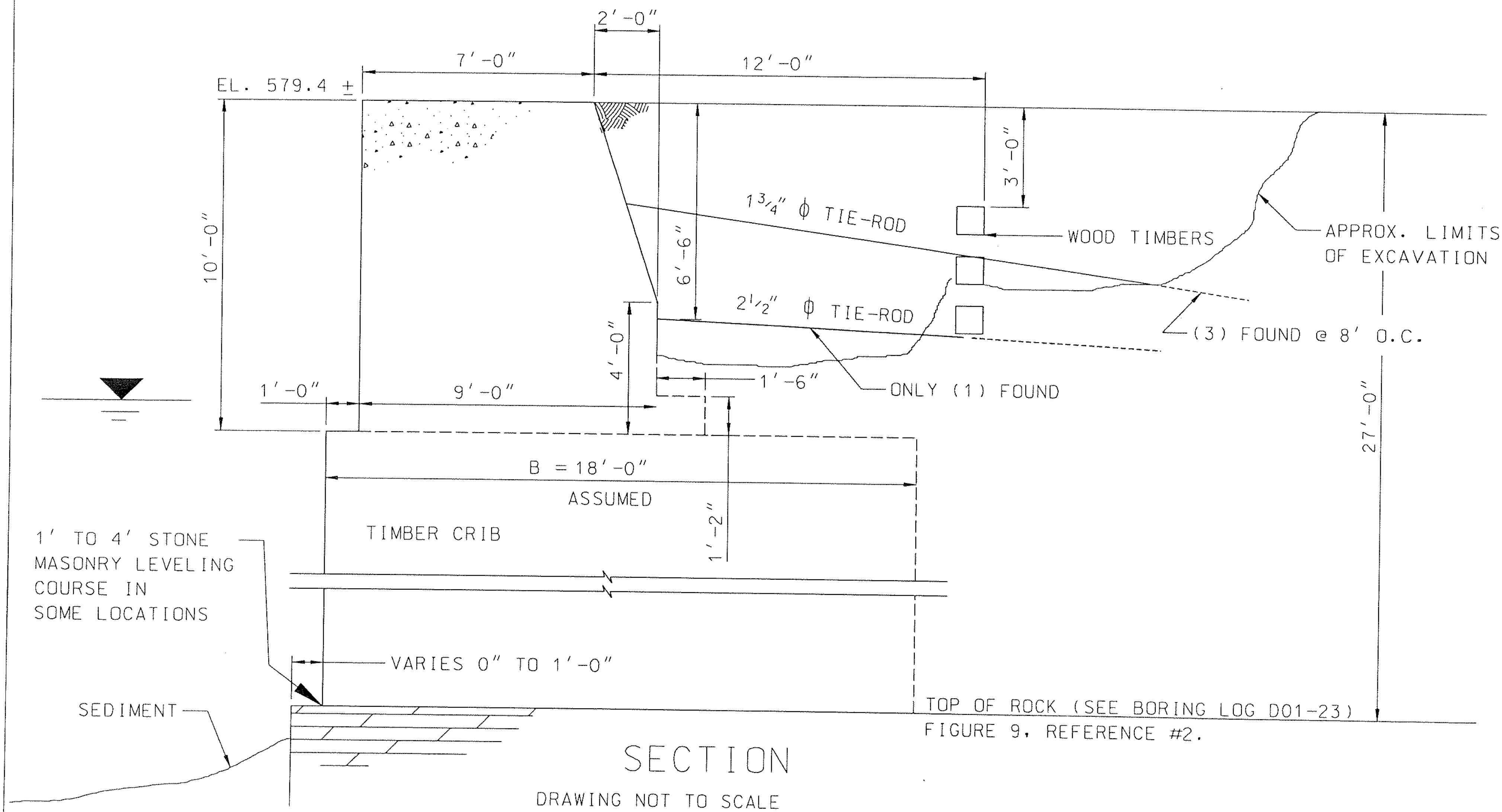
Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 75.46\%$ base in compression < $Pct_{comp\_required} = 100.00\%$ **FAILS**

Note: With a berm, the factor of safety for sliding meets current design requirements and the percent base in compression meets the requirements for existing structures.

UNION SHIP CANAL - REACH H

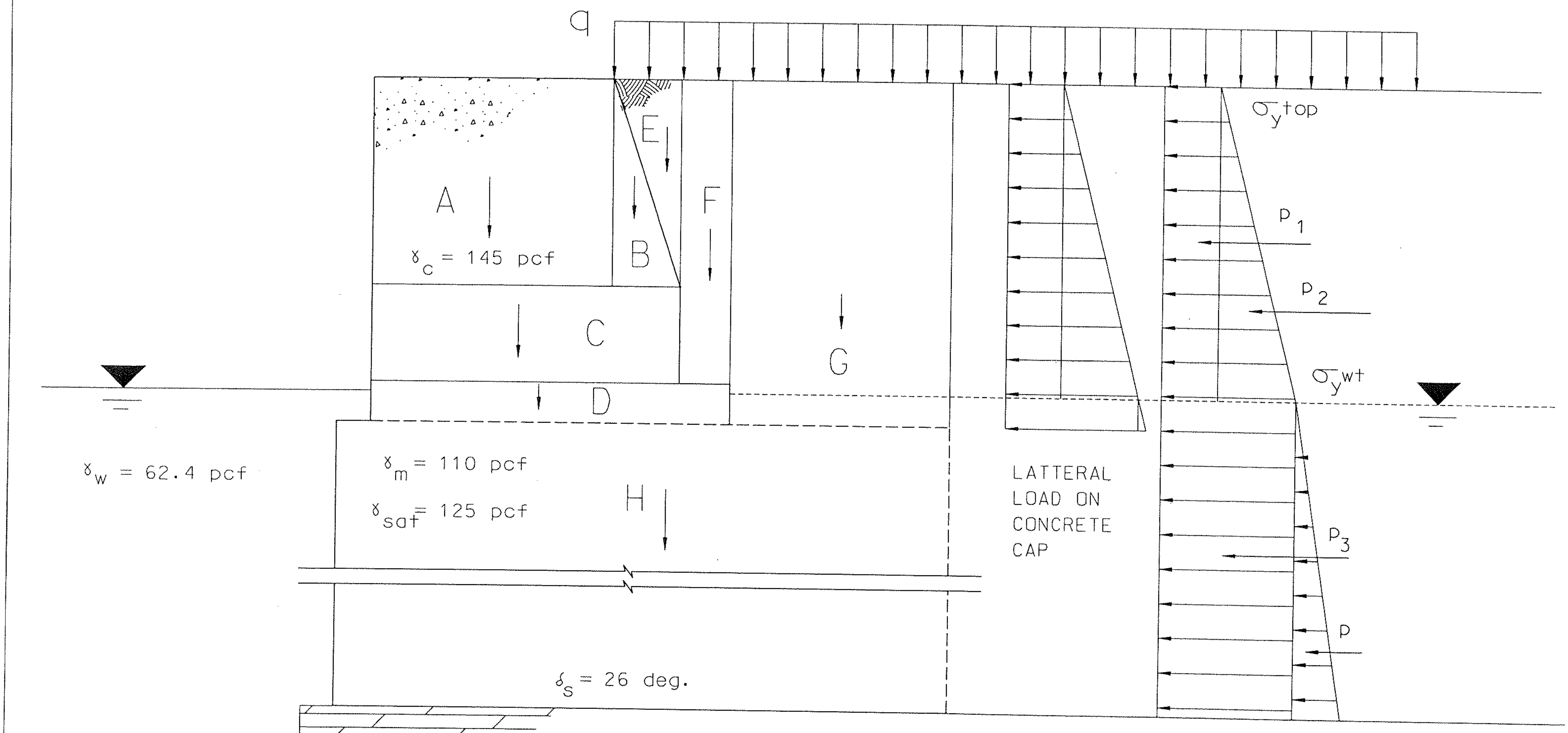
(REPRESENTATIVE OF NORTH WALL STA. 0+00 TO 8+00)



SECTION

DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

UNION SHIP CANAL - REACH H



SECTION

DRAWING NOT TO SCALE

ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

Stability Anal. of Reach H - Top Cap

Material Properties

Water

$\gamma_w := 62.4 \cdot pcf$ Unit Weight of Water

Concrete

$\gamma_c := 145 \cdot pcf$ Unit Weight of Concrete

$f'_c := 6000 \cdot psi$ Compressive Strength of Concrete (Reference 1 - Table 1, Page 3)

Soil

$\gamma_m := 110 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 9)

$\gamma_{sat} := 127 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 9)

$\phi := 31 \cdot deg$ Angle of Internal Friction (Reference 2 - Figure 9)

$\delta_{conc} := 20 \cdot deg$ Angle of Wall Friction (Concrete) (Reference 2 - Figure 9)

$\delta_{crib} := 27 \cdot deg$ Angle of Wall Friction (Cribbing) (Reference 2 - Figure 9)

$\gamma_e := (\gamma_{sat} - \gamma_w)$ Effective Unit Weight of Submerged Fill $\gamma_e = 64.60 pcf$

$\delta_s := 26 \cdot deg$ Shale (fill) Interface Friction Angle (Reference 2 - Figure 9)

Crib Fill

$\gamma_{m\_fill} := 110 \cdot pcf$ Moist Unit Weight (Reference 2 - Figure 9)

$\gamma_{sat\_fill} := 127 \cdot pcf$ Saturated Unit Weight (Reference 2 - Figure 9)

Design Parameters

$SMF := \frac{2}{3}$ Strength Mobilization Factor (Reference 3, Par. 3-13.b)

$\phi_d := atan(SMF \cdot tan(\phi))$ Developed Shear Strength $\phi_d = 21.83 deg$

(Reference 3, eq. 3-10)

$FS_{sliding\_required} := 1.50$ Minimum Sliding Factor of Safety (Reference 3, Table 4-1)

$Pct_{comp\_required} := 100\%$ Minimum Percent Base in Compression (Reference 3, Table 4-1)

Analysis of Top Cap (Ignore stability contributed by tie-rods)

Structure Dimensions

$B := 10.5 \cdot ft$ Base Width

Structure Elevations

$Top := 10 \cdot ft$ Top of Structure

$Base := 0 \cdot ft$ Base of Structure

$W := 1 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf$ Surcharge (Snow)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot B$ $U = 655.20 plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 100.00 psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 1090.00 psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 1154.60 psf$

Latteral Earth Pressure

Horizontal At Rest Earth Pressure Coefficient: $K_O := \left(\tan\left(45 \cdot deg - \frac{\phi_d}{2}\right)\right)^2$ $K_O = 0.46$ (Reference 3, eq. 3-15)

Note: Wall Friction is neglected due to presence of the heel.

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$	Latteral Earth Pressure at Top of Structure	$\sigma_{a\_top} = 45.79\,\text{psf}$
$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$	Latteral Earth Pressure at Water Table	$\sigma_{a\_wt} = 499.10\,\text{psf}$
$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$	Latteral Earth Pressure at Base of Structure	$\sigma_{a\_bot} = 528.67\,\text{psf}$
$P_1 := (\sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_1 = 412.10\,\text{ft}\,\text{psf}$
$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P1} = 5.50\,\text{ft}$
$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_2 = 2039.88\,\text{plf}$
$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P2} = 4.00\,\text{ft}$
$P_3 := \sigma_{a\_wt} \cdot (W - Base)$	Horizontal Pressure	$P_3 = 499.10\,\text{plf}$
$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - Base)\right]$	Resultant Vertical Location	$Y_{P3} = 0.50\,\text{ft}$
$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - Base)$	Horizontal Pressure	$P_4 = 14.79\,\text{plf}$
$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - Base)$	Resultant Vertical Location	$Y_{P4} = 0.33\,\text{ft}$
$P := P_1 + P_2 + P_3 + P_4$	Total Resultant Horizontal Load	$P = 2965.86\,\text{plf}$

$$Y_P := \frac{\left[(P_1 \cdot Y_{P1}) + (P_2 \cdot Y_{P2}) + (P_3 \cdot Y_{P3}) + (P_4 \cdot Y_{P4})\right]}{P} \qquad Y_P = 3.60\,\text{ft}$$

Total Vertical Resultant Location

Weight of Wall

$W_A := (7\cdot ft)\cdot(6\cdot ft)\cdot\gamma_c$ $\qquad W_A = 6090.00\,plf$

$W_B := \left(\frac{1}{2}\right)\cdot(2\cdot ft)\cdot(6\cdot ft)\cdot\gamma_c$ $\qquad W_B = 870.00\,plf$

$W_C := (2.833\cdot ft)\cdot(9\cdot ft)\cdot\gamma_c$ $\qquad W_C = 3697.07\,plf$

$W_D := (1.167\cdot ft)\cdot(10.5\cdot ft)\cdot\gamma_c$ $\qquad W_D = 1776.76\,plf$

$W_E := \left(\frac{1}{2}\right)\cdot(2\cdot ft)\cdot(6\cdot ft)\cdot\gamma_m$ $\qquad W_E = 660.00\,plf$

$W_F := (1.5\cdot ft)\cdot(8.833\cdot ft)\cdot\gamma_m$ $\qquad W_F = 1457.45\,plf$

$Weight := W_A + W_B + W_C + W_D + W_E + W_F$ $\qquad Weight = 14551.27\,plf$

Moment Arms of Elements (About Toe)

$Arm_A := \frac{(7\cdot ft)}{2}$ $\qquad Arm_A = 3.50\,ft$

$Arm_B := (7\cdot ft) + \frac{(2\cdot ft)}{3}$ $\qquad Arm_B = 7.67\,ft$

$Arm_C := \frac{(9\cdot ft)}{2}$ $\qquad Arm_C = 4.50\,ft$

$Arm_D := \frac{(10.5\cdot ft)}{2}$ $\qquad Arm_D = 5.25\,ft$

$Arm_E := (7\cdot ft) + (2\cdot ft)\cdot\frac{2}{3}$ $\qquad Arm_E = 8.33\,ft$

$Arm_F := (9\cdot ft) + \frac{(1.5\cdot ft)}{2}$ $\qquad Arm_F = 9.75\,ft$

Resultant Location of Weight

$$x_W := \frac{W_A\cdot Arm_A + W_B\cdot Arm_B + W_C\cdot Arm_C + W_D\cdot Arm_D + W_E\cdot Arm_E + W_F\cdot Arm_F}{Weight}$$

$x_W = 5.06\,ft$ Horizontal distance from toe to center of mass

Resultant Vertical Stress

$N := Weight - U$ $\qquad$ $N = 13896.07\,plf$

Sliding Resistance

$\tau_{ult} := N \cdot \tan(\delta_s)$ $\qquad$ $\tau_{ult} = 6777.56\,\frac{lb}{ft}$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $\qquad$ $FS_{sliding\_actual} = 2.29$ > $FS_{sliding\_required} = 1.50$ O.K.

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2}\right)}{N}$$ $X_N = 4.28\,ft$

$$e := \left| X_N - \frac{B}{2} \right|$$ $e = 0.97\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B} \right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 100.00\%$ base in compression > $Pct_{comp\_required} = 100.00\%$

Stability Anal. of Reach H - Crib

Material Properties

Water $\gamma_w := 62.4 \cdot pcf$ Unit Weight of Water

Concrete $\gamma_c := 145 \cdot pcf$ Unit Weight of Concrete

$f'_c := 6000 \cdot psi$ Compressive Strength of Concrete (Reference 1 - Table 1, Page 3)

Soil Note: Because of anticipated future use of area surrounding the canal, long term strengths can be assumed for the cohesive materials. Assume a single soil layer, choosing the more conservative value listed for the fill and the clay glacial deposits, for the parameters below:

Note: Soil parameters used for stability of the top cap are for the course grained fill. Soil parameters used for stability of the entire crib are taken from the more critical values of the course grained fill and the predominantly clay glacial deposits. See reference #2.

$\gamma_m := 110 \cdot pcf$	Moist Unit Weight (Fill)	(Reference 2 - Figure 9)
$\gamma_{sat} := 127 \cdot pcf$	Saturated Unit Weight (Fill)	(Reference 2 - Figure 9)
$\phi := 29 \cdot deg$	Angle of Internal Friction (Glacial)	(Reference 2 - Figure 9)
$\delta_{conc} := 20 \cdot deg$	Angle of Wall Friction (Concrete)	(Reference 2 - Figure 9)
$\delta_{crib} := 27 \cdot deg$	Angle of Wall Friction (Cribbing)	(Reference 2 - Figure 9)
$\gamma_e := (\gamma_{sat} - \gamma_w)$	Effective Unit Weight of Submerged Soil	$\gamma_e = 64.60 pcf$
$\delta_s := 26 \cdot deg$	Shale (foundation) Interface Friction Angle	(Reference 2 - Figure 9)

Crib Fill

$\gamma_{m\_fill} := 110 \cdot pcf$	Moist Unit Weight	(Reference 2 - Figure 9)
$\gamma_{sat\_fill} := 127 \cdot pcf$	Saturated Unit Weight	(Reference 2 - Figure 9)

Design Parameters

$SMF := \frac{2}{3}$	Strength Mobilization Factor	(Reference 3, Par. 3-13.b)
$\phi_d := atan(SMF \cdot tan(\phi))$	Developed Shear Strength	$\phi_d = 20.28 deg$ (Reference 3, eq. 3-10)
$FS_{sliding\_required} := 1.50$	Minimum Sliding Factor of Safety	(Reference 3, Table 4-1)
$Pct_{comp\_required} := 100\%$	Minimum Percent Base in Compression	(Reference 3, Table 4-1)

Analysis of Crib

Structure Dimensions

$B := 18 \cdot ft$ Base Width

Structure Elevations

$Top := 27 \cdot ft$ Top of Structure (Reference 1, Boring Log D01-23)

$Base := 0 \cdot ft$ Base of Structure

$W := 18 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf$ Surcharge (Snow)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot B$ $U = 20217.60\,plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 100.00\,psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 1090.00\,psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 2252.80\,psf$

Latteral Earth Pressure

Horizontal At Rest Earth Pressure Coefficient: $K_O := \left(\tan\left(45 \cdot deg - \frac{\phi_d}{2}\right)\right)^2$ $K_O = 0.49$ (Reference 3, eq. 3-15)

Note: Wall Friction is neglected due to irregular profile.

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$	Latteral Earth Pressure at Top of Structure	$\sigma_{a\_top} = 48.52\,psf$
$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$	Latteral Earth Pressure at Water Table	$\sigma_{a\_wt} = 528.86\,psf$
$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$	Latteral Earth Pressure at Base of Structure	$\sigma_{a\_bot} = 1093.04\,psf$
$P_1 := (\sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_1 = 436.67\,ft\,psf$
$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P1} = 22.50\,ft$
$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_2 = 2161.52\,plf$
$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P2} = 21.00\,ft$
$P_3 := \sigma_{a\_wt} \cdot (W - Base)$	Horizontal Pressure	$P_3 = 9519.43\,plf$
$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - Base)\right]$	Resultant Vertical Location	$Y_{P3} = 9.00\,ft$
$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - Base)$	Horizontal Pressure	$P_4 = 5077.61\,plf$
$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - Base)$	Resultant Vertical Location	$Y_{P4} = 6.00\,ft$
$P := P_1 + P_2 + P_3 + P_4$	Total Resultant Horizontal Load	$P = 17195.24\,plf$
$Y_P := \frac{\left[(P_1 \cdot Y_{P1}) + (P_2 \cdot Y_{P2}) + (P_3 \cdot Y_{P3}) + (P_4 \cdot Y_{P4})\right]}{P}$	Total Vertical Resultant Location	$Y_P = 9.97\,ft$

Weight of Wall

$W_A := (7 \cdot ft) \cdot (6 \cdot ft) \cdot \gamma_c$ $\qquad W_A = 6090.00\, plf$

$W_B := \left(\frac{1}{2}\right) \cdot (2 \cdot ft) \cdot (6 \cdot ft) \cdot \gamma_c$ $\qquad W_B = 870.00\, plf$

$W_C := (2.833 \cdot ft) \cdot (9 \cdot ft) \cdot \gamma_c$ $\qquad W_C = 3697.07\, plf$

$W_D := (1.167 \cdot ft) \cdot (10.5 \cdot ft) \cdot \gamma_c$ $\qquad W_D = 1776.76\, plf$

$W_E := \left(\frac{1}{2}\right) \cdot (2 \cdot ft) \cdot (6 \cdot ft) \cdot \gamma_m$ $\qquad W_E = 660.00\, plf$

$W_F := (1.5 \cdot ft) \cdot (8.833 \cdot ft) \cdot \gamma_m$ $\qquad W_F = 1457.45\, plf$

$W_G := (6.5 \cdot ft) \cdot (9 \cdot ft) \cdot \gamma_m + (6.5 \cdot ft) \cdot (1 \cdot ft) \cdot (\gamma_{sat} - \gamma_w)$ $\qquad W_G = 6854.90\, plf$

$W_H := (18 \cdot ft) \cdot (17 \cdot ft) \cdot \gamma_{sat\_fill}$ $\qquad W_H = 38862.00\, plf$

$Weight := W_A + W_B + W_C + W_D + W_E + W_F + W_G + W_H$ $\qquad Weight = 60268.17\, plf$

Moment Arms of Elements (About Toe)

$Arm_A := \frac{(7 \cdot ft)}{2} + 1 \cdot ft$ $\qquad Arm_A = 4.50\, ft$

$Arm_B := (7 \cdot ft) + \frac{(2 \cdot ft)}{3} + 1 \cdot ft$ $\qquad Arm_B = 8.67\, ft$

$Arm_C := \frac{(9 \cdot ft)}{2} + 1 \cdot ft$ $\qquad Arm_C = 5.50\, ft$

$Arm_D := \frac{(10.5 \cdot ft)}{2} + 1 \cdot ft$ $\qquad Arm_D = 6.25\, ft$

$Arm_E := (7 \cdot ft) + (2 \cdot ft) \cdot \frac{2}{3} + 1 \cdot ft$ $\qquad Arm_E = 9.33\, ft$

$Arm_F := (9 \cdot ft) + \frac{(1.5 \cdot ft)}{2} + 1 \cdot ft$ $\qquad Arm_F = 10.75\, ft$

$Arm_G := (10.5 \cdot ft) + \frac{(6.5 \cdot ft)}{2} + 1 \cdot ft$ $\qquad Arm_G = 14.75\, ft$

$Arm_H := \frac{(18 \cdot ft)}{2}$ $\qquad Arm_H = 9.00\, ft$

Resultant Location of Weight

$$X_W := \frac{W_A \cdot Arm_A + W_B \cdot Arm_B + W_C \cdot Arm_C + W_D \cdot Arm_D + W_E \cdot Arm_E + W_F \cdot Arm_F + W_G \cdot Arm_G + W_H \cdot Arm_H}{Weight}$$

$X_W = 8.94\,ft$ Horizontal distance from toe to center of mass

Resultant Vertical Stress

$N := Weight - U$ $N = 40050.57\,plf$

Sliding Resistance

$\tau_{ult} := N \cdot \tan(\delta_s)$ $\tau_{ult} = 19533.97\,\frac{lb}{ft}$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $FS_{sliding\_actual} = 1.14$ < $FS_{sliding\_required} = 1.50$ FAILS

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2}\right)}{N}$$ $X_N = 4.64\,ft$

$$e := \left| X_N - \frac{B}{2} \right|$$ $e = 4.36\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B} \right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 77.30\%$ base in compression < $Pct_{comp\_required} = 100.00\%$ FAILS

Analyse Assuming Tie-Rods are Effective

In the above analyses, the tie-rods were assumed to be ineffective as it was not possible to determine their condition. For comparison, assume that the tie-rods are effective and re-analyse. Additionally, a resisting shear force can be theorized at the interface between the bottom of the cap and the top of the shale crib fill. This shear force would be limited to the lower value of: (1) the excess capacity of the anchors, (2) the sliding resistance of the concrete cap on the shale fill and (3) the capacity of the anchorage. As no information is available on the anchorage, it will be assumed that capacity of the anchorage does not control.

Determine Resisting Force Capacity of Tie-Rods

$A_{tr} := \frac{1}{4}\cdot\pi\cdot(1.5\cdot in)^2$ Area of Tie-Rods $A_{tr} = 1.77\,in^2$

$T_{allow} := A_{tr}\cdot[(0.6)\cdot(36\cdot ksi)]$ Allowable Tension $T_{allow} = 38170.35\,lb$

$Cap_{lat} := \frac{T_{allow}}{8\cdot ft}$ Latteral Capacity on Per Foot Basis $Cap_{lat} = 4771.29\,plf$

Note: Assume A36, very conservative assumption. Strength most likely much greater.

Determine Required Load in Tie-Rods for Sliding Stability

$F_{TR\_sliding} := 4771.29\cdot plf$ Assumed load in tie-rod, itterated by trial and error, to meet sliding F.S., limited to latteral strength capacity of the tie-rods calculated above.

$\tau_{ult} := N\cdot\tan(\delta_s) + F_{TR\_sliding}$ $\tau_{ult} = 24305.26\,\frac{lb}{ft}$ (Reference 3, eq. 4-5)

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $FS_{sliding\_actual} = 1.41$ < $FS_{sliding\_required} = 1.50$ FAILS

Determine Required Load in Tie-Rods for Overturning Stability

$F_{TR\_ovt} := 2273\cdot plf$ Assumed load in tie-rod, itterated by trial and error, to meet overturning F.S.

$$X_N := \frac{\left[Weight\cdot X_W + F_{TR\_ovt}\cdot(24\cdot ft) - P\cdot Y_P - U\cdot\frac{B}{2}\right]}{N}$$ $X_N = 6.00\,ft$

$$e := \left|X_N - \frac{B}{2}\right|$$ $e = 3.00\,ft$

$$Pct_{comp\_actual} := \mathrm{if}\left[e < \frac{B}{6}, 1.00, 3\cdot\frac{\left(\frac{B}{2} - e\right)}{B}\right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 100.00\,\%$ base in compression > $Pct_{comp\_required} = 100.00\,\%$ O.K.

Controlling Tie-Rod Load

$F_{TR} := \max(F_{TR\_sliding}, F_{TR\_ovt})$ $F_{TR} = 4771.29\,\frac{lb}{ft}$

Check F.S. Against Bulging (Reference 4, Par. 5.7.2)

$P'_a := 2965.86 \cdot plf$ Resultant Horizontal Load on Concrete Cap, from Calculations in "H Top"

$F_g := F_{TR}$ Horizontal Load in Tie-Rods as Required

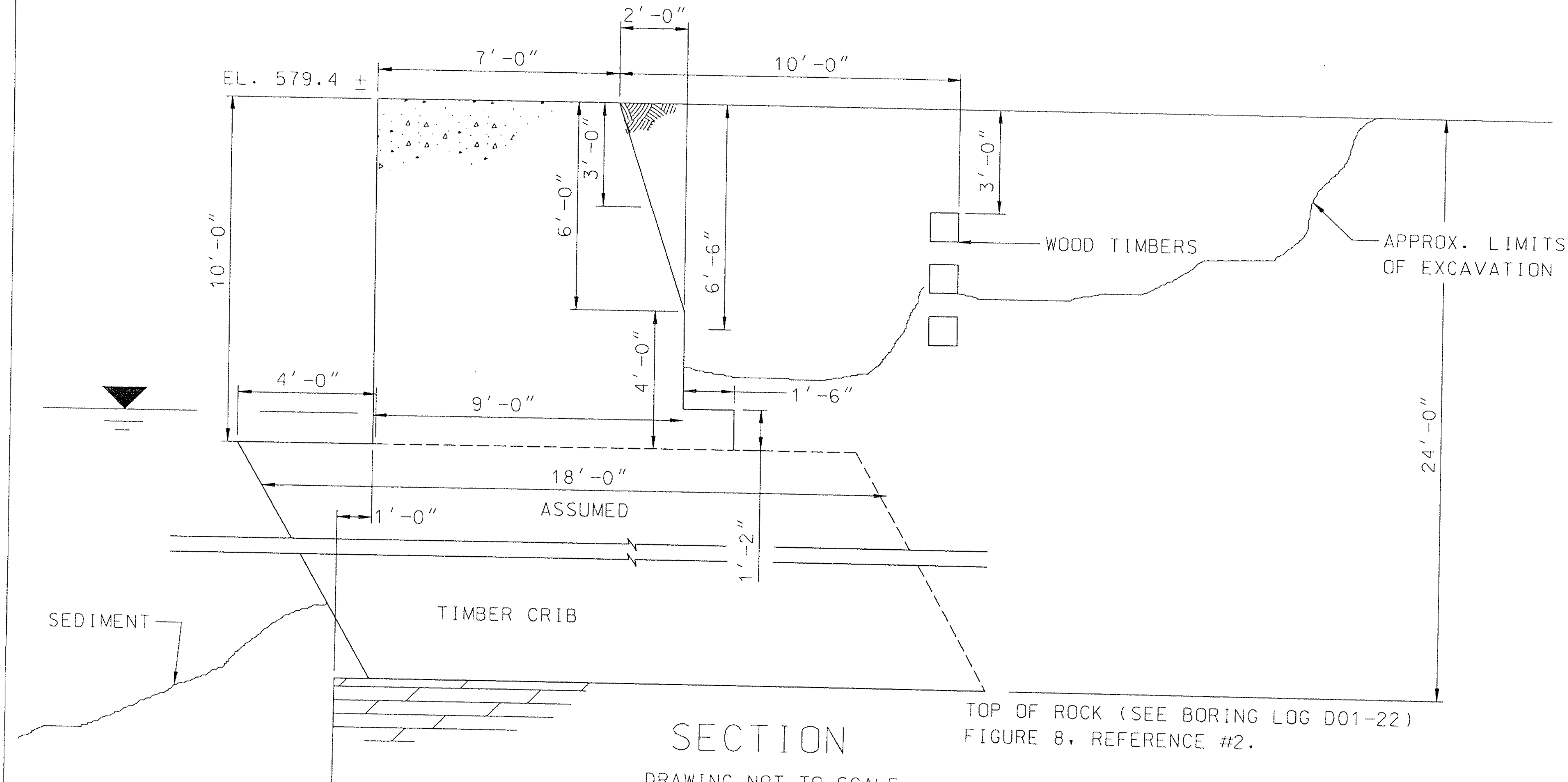
$V_h := 6777.56 \cdot plf$ Sliding Resistance of Concrete Cap on Shale Crib Fill, from Calculations in "H Top"

$FS_{bulging} := \frac{V_h}{\left|P'_a - F_g\right|}$ Factor of Safety Against Bulging $FS_{bulging} = 3.75$ > 1.5 O.K.

Note: Though the required factor of safety for sliding is not met for a new structure and normal load condition, the factor of safety for sliding for an unusual condition is met (1.33). Assumptions made for the analysis are conservative, i.e. analysis is for conservative steel strength values, any passive soil has been neglected, most conservative value for each of the soil parameters was used, snow load does not occur over most of the year but is considered a normal load condition. As there is no evidence of instability in the wall (visible misalignment, rotation or translation), for the above reasons, the stability of the wall is considered adequate for the anticipated future uses and loads by engineering judgement.

UNION SHIP CANAL - REACH I

(REPRESENTATIVE OF NORTH WALL STA. 8+00 TO 10+00)

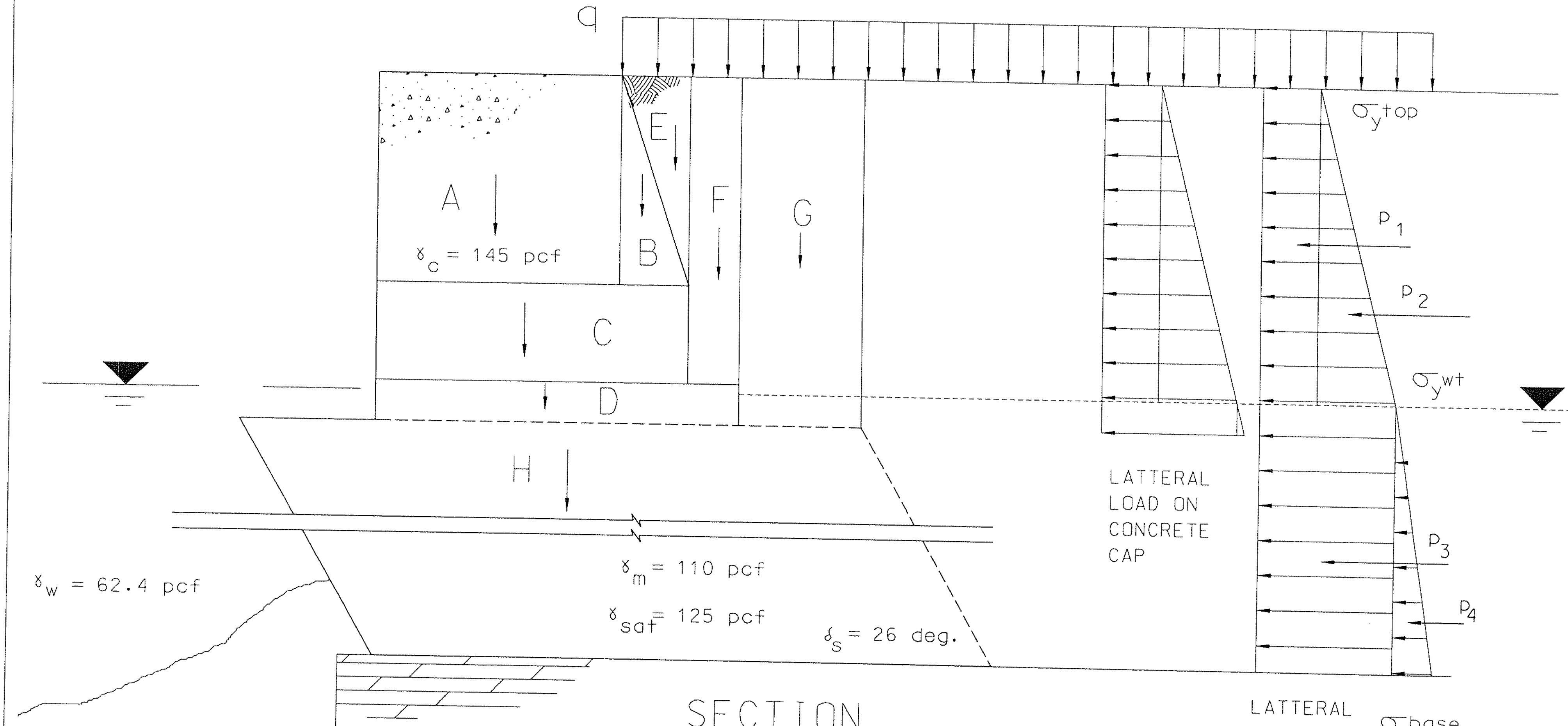


SECTION

DRAWING NOT TO SCALE

ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

UNION SHIP CANAL - REACH I



SECTION

DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

Stability Anal. of Reach I - Top Cap

Material Properties

Water

$\gamma_w := 62.4 \cdot pcf$ Unit Weight of Water

Concrete

$\gamma_c := 145 \cdot pcf$ Unit Weight of Concrete

$f'_c := 6000 \cdot psi$ Compressive Strength of Concrete (Reference 1 - Table 1, Page 3)

Soil

Note: Soil parameters used for stability of the top cap are for the course grained fill. Soil parameters used for stability of the entire crib are taken from the more critical values of the course grained fill and the predominantly clay glacial deposits. See reference #2.

$\gamma_m := 110 \cdot pcf$	Moist Unit Weight	(Reference 2 - Figure 9)
$\gamma_{sat} := 127 \cdot pcf$	Saturated Unit Weight	(Reference 2 - Figure 9)
$\phi := 31 \cdot deg$	Angle of Internal Friction	(Reference 2 - Figure 9)
$\delta_{conc} := 20 \cdot deg$	Angle of Wall Friction (Concrete)	(Reference 2 - Figure 9)
$\delta_{crib} := 27 \cdot deg$	Angle of Wall Friction (Cribbing)	(Reference 2 - Figure 9)
$\gamma_e := (\gamma_{sat} - \gamma_w)$	Effective Unit Weight of Submerged Fill	$\gamma_e = 64.60 pcf$
$\delta_s := 26 \cdot deg$	Shale (fill) Interface Friction Angle	(Reference 2 - Figure 9)

Crib Fill

$\gamma_{m\_fill} := 110 \cdot pcf$	Moist Unit Weight	(Reference 2 - Figure 9)
$\gamma_{sat\_fill} := 127 \cdot pcf$	Saturated Unit Weight	(Reference 2 - Figure 9)

Design Parameters

$SMF := \frac{2}{3}$	Strength Mobilization Factor	(Reference 3, Par. 3-13.b)
$\phi_d := atan(SMF \cdot tan(\phi))$	Developed Shear Strength	$\phi_d = 21.83 deg$ (Reference 3, eq. 3-10)
$FS_{sliding\_required} := 1.50$	Minimum Sliding Factor of Safety	(Reference 3, Table 4-1)
$Pct_{comp\_required} := 100\%$	Minimum Percent Base in Compression	(Reference 3, Table 4-1)

Analysis of Top Cap (Ignore stability contributed by tie-rods)

Structure Dimensions

$B := 10.5 \cdot ft$ Base Width

Structure Elevations

$Top := 10 \cdot ft$ Top of Structure

$Base := 0 \cdot ft$ Base of Structure

$W := 1 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf$ Surcharge (Snow)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot B$ $U = 655.20\,plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 100.00\,psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 1090.00\,psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 1154.60\,psf$

Latteral Earth Pressure

Horizontal At Rest Earth Pressure Coefficient: $K_O := \left(\tan\left(45 \cdot deg - \frac{\phi_d}{2}\right)\right)^2$ $K_O = 0.46$ (Reference 3, eq. 3-15)

Note: Wall Friction is neglected due to presence of the heel.

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$	Latteral Earth Pressure at Top of Structure	$\sigma_{a\_top} = 45.79\,psf$
$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$	Latteral Earth Pressure at Water Table	$\sigma_{a\_wt} = 499.10\,psf$
$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$	Latteral Earth Pressure at Base of Structure	$\sigma_{a\_bot} = 528.67\,psf$
$P_1 := (\sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_1 = 412.10\,ft\,psf$
$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P1} = 5.50\,ft$
$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_2 = 2039.88\,plf$
$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P2} = 4.00\,ft$
$P_3 := \sigma_{a\_wt} \cdot (W - Base)$	Horizontal Pressure	$P_3 = 499.10\,plf$
$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - Base)\right]$	Resultant Vertical Location	$Y_{P3} = 0.50\,ft$
$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - Base)$	Horizontal Pressure	$P_4 = 14.79\,plf$
$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - Base)$	Resultant Vertical Location	$Y_{P4} = 0.33\,ft$
$P := P_1 + P_2 + P_3 + P_4$	Total Resultant Horizontal Load	$P = 2965.86\,plf$
$Y_P := \frac{\left[(P_1 \cdot Y_{P1}) + (P_2 \cdot Y_{P2}) + (P_3 \cdot Y_{P3}) + (P_4 \cdot Y_{P4})\right]}{P}$	Total Vertical Resultant Location	$Y_P = 3.60\,ft$

Weight of Wall

$W_A := (7 \cdot ft) \cdot (6 \cdot ft) \cdot \gamma_c$ $\quad W_A = 6090.00\,plf$

$W_B := \left(\frac{1}{2}\right) \cdot (2 \cdot ft) \cdot (6 \cdot ft) \cdot \gamma_c$ $\quad W_B = 870.00\,plf$

$W_C := (2.833 \cdot ft) \cdot (9 \cdot ft) \cdot \gamma_c$ $\quad W_C = 3697.07\,plf$

$W_D := (1.167 \cdot ft) \cdot (10.5 \cdot ft) \cdot \gamma_c$ $\quad W_D = 1776.76\,plf$

$W_E := \left(\frac{1}{2}\right) \cdot (2 \cdot ft) \cdot (6 \cdot ft) \cdot \gamma_m$ $\quad W_E = 660.00\,plf$

$W_F := (1.5 \cdot ft) \cdot (8.833 \cdot ft) \cdot \gamma_m$ $\quad W_F = 1457.45\,plf$

$Weight := W_A + W_B + W_C + W_D + W_E + W_F$ $\quad Weight = 14551.27\,plf$

Moment Arms of Elements (About Toe)

$Arm_A := \frac{(7 \cdot ft)}{2}$ $\quad Arm_A = 3.50\,ft$

$Arm_B := (7 \cdot ft) + \frac{(2 \cdot ft)}{3}$ $\quad Arm_B = 7.67\,ft$

$Arm_C := \frac{(9 \cdot ft)}{2}$ $\quad Arm_C = 4.50\,ft$

$Arm_D := \frac{(10.5 \cdot ft)}{2}$ $\quad Arm_D = 5.25\,ft$

$Arm_E := (7 \cdot ft) + (2 \cdot ft) \cdot \frac{2}{3}$ $\quad Arm_E = 8.33\,ft$

$Arm_F := (9 \cdot ft) + \frac{(1.5 \cdot ft)}{2}$ $\quad Arm_F = 9.75\,ft$

Resultant Location of Weight

$$X_W := \frac{W_A \cdot Arm_A + W_B \cdot Arm_B + W_C \cdot Arm_C + W_D \cdot Arm_D + W_E \cdot Arm_E + W_F \cdot Arm_F}{Weight}$$

$X_W = 5.06\,ft$ Horizontal distance from toe to center of mass

Resultant Vertical Stress

$N := \text{Weight} - U$ $\qquad N = 13896.07\,\text{plf}$

Sliding Resistance

$\tau_{ult} := N \cdot \tan(\delta_s)$ $\qquad \tau_{ult} = 6777.56\,\frac{\text{lb}}{\text{ft}}$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $\qquad FS_{sliding\_actual} = 2.29 \quad > \quad FS_{sliding\_required} = 1.50$ O.K.

Resultant location (from center of base)

$$X_N := \frac{\left(\text{Weight} \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2}\right)}{N}$$ $\qquad X_N = 4.28\,\text{ft}$

$$e := \left| X_N - \frac{B}{2} \right|$$ $\qquad e = 0.97\,\text{ft}$

$$Pct_{comp\_actual} := \text{if}\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B} \right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 100.00\%$ base in compression $\quad > \quad Pct_{comp\_required} = 100.00\%$

Stability Anal. of Reach I - Crib

Material Properties

Water

$\gamma_w := 62.4 \cdot pcf$ — Unit Weight of Water

Concrete

$\gamma_c := 145 \cdot pcf$ — Unit Weight of Concrete

$f'_c := 6000 \cdot psi$ — Compressive Strength of Concrete (Reference 1 - Table 1, Page 3)

Soil

Note: Because of anticipated future use of area surrounding the canal, long term strengths can be assumed for the cohesive materials. Assume a single soil layer, choosing the more conservative value listed for the fill and the clay glacial deposits, for the parameters below:

$\gamma_m := 110 \cdot pcf$	Moist Unit Weight (Fill)	(Reference 2 - Figure 8)
$\gamma_{sat} := 127 \cdot pcf$	Saturated Unit Weight (Fill)	(Reference 2 - Figure 8)
$\phi := 29 \cdot deg$	Angle of Internal Friction (Glacial)	(Reference 2 - Figure 8)
$\delta_{conc} := 20 \cdot deg$	Angle of Wall Friction (Concrete)	(Reference 2 - Figure 8)
$\delta_{crib} := 27 \cdot deg$	Angle of Wall Friction (Cribbing)	(Reference 2 - Figure 8)
$\gamma_e := (\gamma_{sat} - \gamma_w)$	Effective Unit Weight of Submerged Soil	$\gamma_e = 64.60 pcf$
$\delta_s := 26 \cdot deg$	Shale (foundation) Interface Friction Angle	(Reference 2 - Figure 8)

Crib Fill

$\gamma_{m\_fill} := 110 \cdot pcf$	Moist Unit Weight	(Reference 2 - Figure 8)
$\gamma_{sat\_fill} := 127 \cdot pcf$	Saturated Unit Weight	(Reference 2 - Figure 8)

Design Parameters

$SMF := \frac{2}{3}$	Strength Mobilization Factor	(Reference 3, Par. 3-13.b)
$\phi_d := atan(SMF \cdot tan(\phi))$	Developed Shear Strength	$\phi_d = 20.28 deg$ (Reference 3, eq. 3-10)
$FS_{sliding\_required} := 1.50$	Minimum Sliding Factor of Safety	(Reference 3, Table 4-1)
$Pct_{comp\_required} := 100\%$	Minimum Percent Base in Compression	(Reference 3, Table 4-1)

Analysis of Crib

Structure Dimensions

$B := 18 \cdot ft$ Base Width

Structure Elevations

$Top := 24 \cdot ft$ Top of Structure (Reference 1, Boring Log D01-16)

$Base := 0 \cdot ft$ Base of Structure

$W := 15 \cdot ft$ Water Level Assumed equal elevations in canal and backfill.

Loading Conditions

$q := 100 \cdot psf$ Surcharge (Snow)

Hydrostatic Water Forces

Note: Lateral hydrostatic forces cancel due to equal water elevations on both sides of the structure.

Hydrostatic Uplift at Base: $U := \gamma_w \cdot (W - Base) \cdot B$ $U = 16848.00\,plf$

Vertical Stresses at Ground Surface

Vertical Stress at Ground Surface: $\sigma_{ytop} := q$ $\sigma_{ytop} = 100.00\,psf$

Vertical Stresses at Water Table

Vertical Stress at Water Table: $\sigma_{ywt} := \gamma_m \cdot (Top - W) + q$ $\sigma_{ywt} = 1090.00\,psf$

Vertical Stress at Base

Vertical Stress at Base: $\sigma_{ybase} := \sigma_{ywt} + \gamma_e \cdot (W - Base)$ $\sigma_{ybase} = 2059.00\,psf$

Latteral Earth Pressure

Horizontal At Rest Earth Pressure Coefficient: $K_O := \left(\tan\left(45 \cdot deg - \frac{\phi_d}{2}\right)\right)^2$ $K_O = 0.49$ (Reference 3, eq. 3-15)

Note: Wall Friction is neglected due to irregular profile.

$\sigma_{a\_top} := K_O \cdot (\sigma_{ytop})$	Latteral Earth Pressure at Top of Structure	$\sigma_{a\_top} = 48.52\,psf$
$\sigma_{a\_wt} := K_O \cdot (\sigma_{ywt})$	Latteral Earth Pressure at Water Table	$\sigma_{a\_wt} = 528.86\,psf$
$\sigma_{a\_bot} := K_O \cdot (\sigma_{ybase})$	Latteral Earth Pressure at Base of Structure	$\sigma_{a\_bot} = 999.01\,psf$
$P_1 := (\sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_1 = 436.67\,ft\,psf$
$Y_{P1} := W + \left[\left(\frac{1}{2}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P1} = 19.50\,ft$
$P_2 := .5 \cdot (\sigma_{a\_wt} - \sigma_{a\_top}) \cdot (Top - W)$	Horizontal Pressure	$P_2 = 2161.52\,plf$
$Y_{P2} := W + \left[\left(\frac{1}{3}\right) \cdot (Top - W)\right]$	Resultant Vertical Location	$Y_{P2} = 18.00\,ft$
$P_3 := \sigma_{a\_wt} \cdot (W - Base)$	Horizontal Pressure	$P_3 = 7932.86\,plf$
$Y_{P3} := \left[\left(\frac{1}{2}\right) \cdot (W - Base)\right]$	Resultant Vertical Location	$Y_{P3} = 7.50\,ft$
$P_4 := 0.5 \cdot (\sigma_{a\_bot} - \sigma_{a\_wt}) \cdot (W - Base)$	Horizontal Pressure	$P_4 = 3526.12\,plf$
$Y_{P4} := \left(\frac{1}{3}\right) \cdot (W - Base)$	Resultant Vertical Location	$Y_{P4} = 5.00\,ft$
$P := P_1 + P_2 + P_3 + P_4$	Total Resultant Horizontal Load	$P = 14057.18\,plf$
$Y_P := \frac{\left[(P_1 \cdot Y_{P1}) + (P_2 \cdot Y_{P2}) + (P_3 \cdot Y_{P3}) + (P_4 \cdot Y_{P4})\right]}{P}$	Total Vertical Resultant Location	$Y_P = 8.86\,ft$

Weight of Wall

$W_A := (7\cdot ft)\cdot(6\cdot ft)\cdot\gamma_c$ $\quad W_A = 6090.00\,plf$

$W_B := \left(\frac{1}{2}\right)\cdot(2\cdot ft)\cdot(6\cdot ft)\cdot\gamma_c$ $\quad W_B = 870.00\,plf$

$W_C := (2.833\cdot ft)\cdot(9\cdot ft)\cdot\gamma_c$ $\quad W_C = 3697.07\,plf$

$W_D := (1.167\cdot ft)\cdot(10.5\cdot ft)\cdot\gamma_c$ $\quad W_D = 1776.76\,plf$

$W_E := \left(\frac{1}{2}\right)\cdot(2\cdot ft)\cdot(6\cdot ft)\cdot\gamma_m$ $\quad W_E = 660.00\,plf$

$W_F := (1.5\cdot ft)\cdot(8.833\cdot ft)\cdot\gamma_m$ $\quad W_F = 1457.45\,plf$

$W_G := (6.5\cdot ft)\cdot(9\cdot ft)\cdot\gamma_m + (6.5\cdot ft)\cdot(1\cdot ft)\cdot(\gamma_{sat} - \gamma_w)$ $\quad W_G = 6854.90\,plf$

$W_H := (18\cdot ft)\cdot(14\cdot ft)\cdot\gamma_{sat\_fill}$ $\quad W_H = 32004.00\,plf$

$Weight := W_A + W_B + W_C + W_D + W_E + W_F + W_G + W_H$ $\quad Weight = 53410.17\,plf$

Moment Arms of Elements (About Toe)

$Arm_A := \frac{(7\cdot ft)}{2} + 1\cdot ft$ $\quad Arm_A = 4.50\,ft$

$Arm_B := (7\cdot ft) + \frac{(2\cdot ft)}{3} + 1\cdot ft$ $\quad Arm_B = 8.67\,ft$

$Arm_C := \frac{(9\cdot ft)}{2} + 1\cdot ft$ $\quad Arm_C = 5.50\,ft$

$Arm_D := \frac{(10.5\cdot ft)}{2} + 1\cdot ft$ $\quad Arm_D = 6.25\,ft$

$Arm_E := (7\cdot ft) + (2\cdot ft)\cdot\frac{2}{3} + 1\cdot ft$ $\quad Arm_E = 9.33\,ft$

$Arm_F := (9\cdot ft) + \frac{(1.5\cdot ft)}{2} + 1\cdot ft$ $\quad Arm_F = 10.75\,ft$

$Arm_G := (10.5\cdot ft) + \frac{(6.5\cdot ft)}{2} + 1\cdot ft$ $\quad Arm_G = 14.75\,ft$

$Arm_H := \frac{(18\cdot ft)}{2} - 2\cdot ft$ $\quad Arm_H = 7.00\,ft$

Resultant Location of Weight

$$X_W := \frac{W_A \cdot Arm_A + W_B \cdot Arm_B + W_C \cdot Arm_C + W_D \cdot Arm_D + W_E \cdot Arm_E + W_F \cdot Arm_F + W_G \cdot Arm_G + W_H \cdot Arm_H}{Weight}$$

$X_W = 7.74\,ft$ Horizontal distance from toe to center of mass

Resultant Vertical Stress

$N := Weight - U$ $N = 36562.17\,plf$

Sliding Resistance

$\tau_{ult} := N \cdot \tan(\delta_s)$ $\tau_{ult} = 17832.56\,\frac{lb}{ft}$ (Reference 3, eq. 4-5)

Sliding Factor of Safety

$FS_{sliding\_actual} := \frac{\tau_{ult}}{P}$ $FS_{sliding\_actual} = 1.27$ < $FS_{sliding\_required} = 1.50$ Fails

Resultant location (from center of base)

$$X_N := \frac{\left(Weight \cdot X_W - P \cdot Y_P - U \cdot \frac{B}{2}\right)}{N}$$ $X_N = 3.75\,ft$

$$e := \left| X_N - \frac{B}{2} \right|$$ $e = 5.25\,ft$

$$Pct_{comp\_actual} := if\left[e < \frac{B}{6}, 1.00, 3 \cdot \frac{\left(\frac{B}{2} - e\right)}{B} \right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 62.53\,\%$ base in compression < $Pct_{comp\_required} = 100.00\,\%$ Fails

If stabilization methods were implemented, what would be the necessary height of the passive soil?

Note: Assume a stone berm is placed. Stone will be placed on top of silt, silt will not be removed, so silt will act as part of berm. Use parameters of 118 pcf and $\phi = 30$ deg for stone. As both of these parameters are similar to those of the silt, it is conservative to use the parameters for the silt for the design. Assume the slope of the berm is set for 1 on 2 or 26.5 degrees below horizontal.

$h_{silt} := 14 \cdot ft$ $\beta_{berm} := -9 \cdot deg$

$\gamma_{e\_silt\_sat} := 115 \cdot pcf - \gamma_w$ $\gamma_{e\_silt\_sat} = 52.60 pcf$

$\phi_{silt} := 27 \cdot deg$

Note: SMF is not used for calculating passive earth pressure coefficients per Reference 3, Para. 3-13.b.

Vertical Stress at Base - Passive Side

Vertical Stress at Base: $\sigma_{ybase\_silt} := \gamma_{e\_silt\_sat} \cdot (h_{silt})$ $\sigma_{ybase\_silt} = 736.40 psf$

Horizontal At Rest Earth Pressure Coefficient:

$$K_p := \frac{(\cos(\phi_{silt}))^2}{\left(1 - \sqrt{\frac{\sin(\phi_{silt}) \cdot \sin(\phi_{silt} + \beta_{berm})}{\cos(\beta_{berm})}}\right)^2}$$

$K_p = 2.04$ (Reference 3, eq. 3-20)

Note: Wall Friction is neglected due to irregular profile.

Latteral Earth Pressure at Base of Structure $\sigma_{p\_bot\_silt} := K_p \cdot (\sigma_{ybase\_silt})$ $\sigma_{p\_bot\_silt} = 1505.68 psf$

Horizontal Pressure $P_{silt} := .5 \cdot (\sigma_{p\_bot\_silt}) \cdot (h_{silt})$ $P_{silt} = 10539.79 ft psf$

Resultant Vertical Location $Y_{Psilt} := \frac{(h_{silt})}{3}$ $Y_{Psilt} = 4.67 ft$

Sliding Factor of Safety

$$FS_{sliding\_actual} := \frac{\tau_{ult}}{P - \left(\frac{P_{silt}}{2}\right)}$$

$FS_{sliding\_actual} = 2.03$

$> FS_{sliding\_required} = 1.50$ O.K.

Note: Reference 3, Para. 3-8.b limits passive resistance for stability calculations to 1/2 that of calculated passive resistance.

Resultant location (from center of base)

$$X_N := \frac{\left(\text{Weight}\cdot X_W - P\cdot Y_P - U\cdot\frac{B}{2} + \frac{P_{silt}}{2}\cdot Y_{Psilt}\right)}{N}$$

$X_N = 4.42\,\text{ft}$

$$e := \left|X_N - \frac{B}{2}\right|$$

$e = 4.58\,\text{ft}$

$$Pct_{comp\_actual} := \text{if}\left[e < \frac{B}{6}, 1.00, 3\cdot\frac{\left(\frac{B}{2} - e\right)}{B}\right]$$

Calculates percent base in compression based on location of the resultant.

$Pct_{comp\_actual} = 73.74\%$ base in compression $<$ $Pct_{comp\_required} = 100.00\%$ **FAILS**

Note: 73% base in compression is close to the allowable of 75% for existing structures. Based on the conservative assumptions made for the load case and by judgement, consider acceptable. A slope of (-9) deg. is the natural slope that the soil would be stable at under water, see slope stability calculation. No benefit would be derived by using rock to make the angle steeper. Use dredged material.

US Army Corps of Engineers
Buffalo District

Union Ship Canal
Buffalo, New York

Cost Estimate

Appendix D

November 2002

Union Ship Canal
Cost Estimate

Cost Estimates were developed for the alternatives described under the Structural Appendix. A total of 4 alternatives were developed and estimated. The alternatives are sand slope stabilization, stone slope stabilization, Steel sheet Pile Wall with fill behind the wall, and Stone Fill. Refer to the plan views at the end of this appendix.

Assumptions:

1. All work will be completed using land-based equipment.
2. Clearing and Grubbing will be required for all alternatives
3. A Bike Path and Walkway will be constructed as part of each alternative
4. Areas will be receive topsoil and seed
5. The large discharge pipe currently penetrating into the north east corner of the canal will require extension
6. Unit Prices
   a. Abstracted from like work, reviewed and adjusted for this job, or
   b. Prices were developed using plant, labor and material cost estimating techniques

Items of like nature to all estimates:

Clearing & Grubbing – Removal of trees and shrubs along the canal has been deemed necessary to construct any of these alternatives

Removals – Concrete and Soil Removal is deemed necessary to develop a smooth transition between the surrounding land and the new work. This item has been added to all alternatives.

Pipe Extension – The large diameter discharge pipe at the northeast portion of the canal will be extended under all alternatives. Depending on the alternative the pipe will have to be extended only out past the stone or sand stabilization or all the way past the SSP, or end of the fill in these alternatives.

Bike Path and Walkway – A plan view of a Bike Path and Walkway was developed and estimated. This feature includes concrete sidewalks, asphalt bike path and railings.

Unique Items within each estimate

Sand Stabilization – A sand Stabilization berm covered with a riprap slope between –3 LWD and +8LWD estimate was developed. Sand would be purchased and trucked to the site and placed to the lines and grades outlined. A riprap cover would be added to stabilize the slope.

Stone Stabilization – Same as the above, however stone would be purchased and placed. No cover of riprap would be required.

Steel Sheet Pile Wall with Fill – A Steel Sheet Pile (SSP) Wall would be built across the canal using PZ27 with a wale and rock anchor system. Fill would be placed behind the SSP to an elevation of 577.0. The SSP wall would be approx 1200 feet from the eastern end of the canal. At this stage of development it is thought that contractor will be able to build a causeway across the canal and work from this instead of bringing in marine plant. A stone berm will be required on the west side of the SSP to help stabilize the wall.

Partially Filled Canal - Contractor shall fill the canal from the eastern end approximately 1200 feet westward. Fill shall be a stone fill and be placed and compacted to an elevation of 577.

All estimates contain a contingency of 25%.

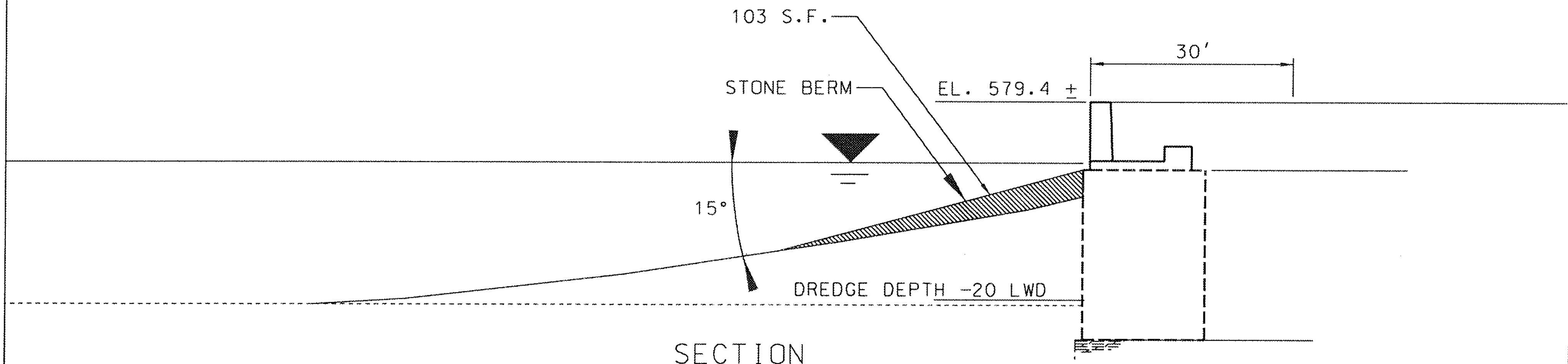
UNION SHIP CANAL, BUFFALO, NEW YORK
PLANNING ASSISTANCE TO STATES

COST ESTIMATE
STONE SLOPE STABILZATION

FEATURE	QUANTITY	UNIT OF MEASURE	CONTRACT	CONTINGENCY	TOTAL COST
REMOVALS			$114,314	$28,578	$142,892
CLEARING & GRUBBING	3	ACRE	$20,560	$5,140	$25,701
STONE STABILIZATION	6600	TON	$2,244,000	$224,400	$2,468,400
WALK-WAY	1	EACH	$881,106	$220,277	$1,101,383
RIPRAPED SLOPE	12500	TON	$503,125	$125,781	$628,906
PIPE EXTENSION			$11,310	$2,828	$14,138
			$3,774,415	$607,004	$4,381,420

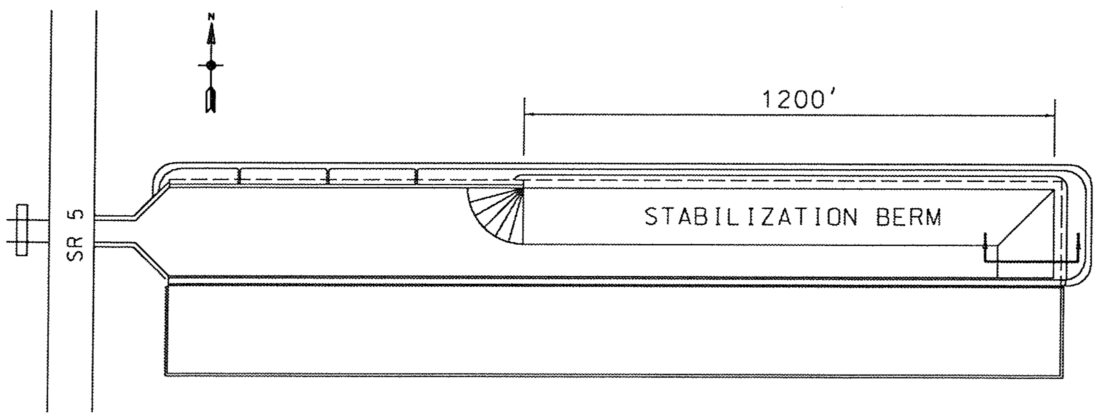
STONE SLOPE STABILIZATION OPTION

REACH D - SOUTH 100 L.F. OF EAST END WALL



SECTION

GENERAL LAYOUT SLOPE STABILIZATION OPTION



NOTES:

1. FILL MATERIAL TO BE STONE

UNION SHIP CANAL

UNION SHIP CANAL
WALL STABILIZATION OPTIONS

DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

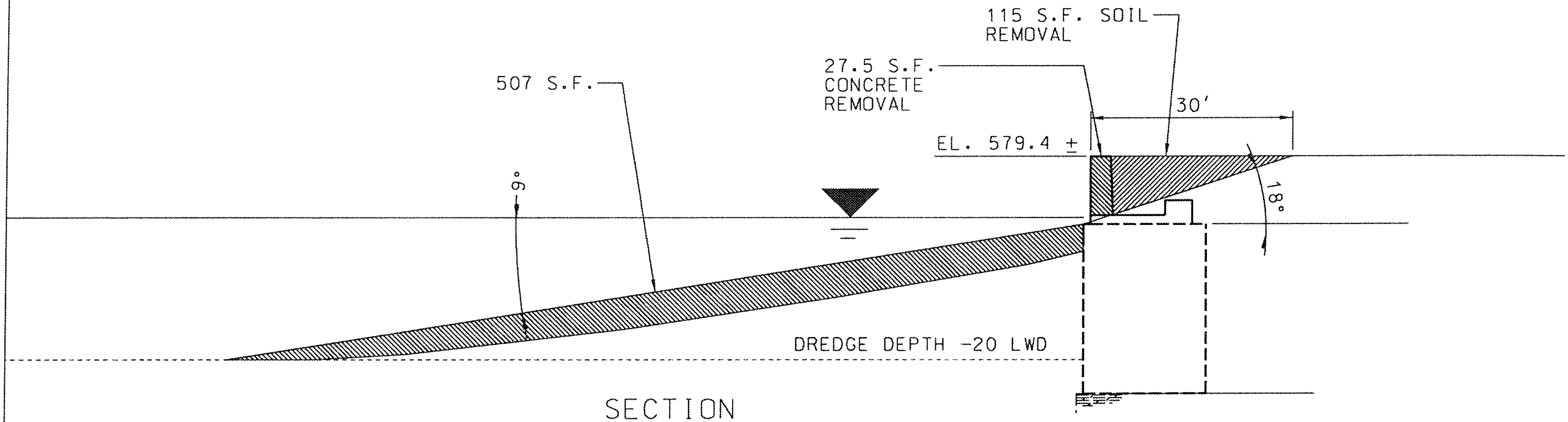
UNION SHIP CANAL, BUFFALO, NEW YORK
PLANNING ASSISTANCE TO STATES

COST ESTIMATE
SAND SLOPE STABILZATION

FEATURE	QUANTITY	UNIT OF MEASURE	CONTRACT	CONTINGENCY	TOTAL COST
REMOVALS			$114,314	$28,578	$142,892
CLEARING & GRUBBING	3	ACRE	$20,560	$5,140	$25,701
WALK-WAY	1	EACH	$881,106	$220,277	$1,101,383
SAND STABILIZATION	67750	TON	$911,238	$91,124	$1,002,361
RIPRAPED SLOPE	12500	TON	$503,125	$50,313	$553,438
PIPE EXTENSION			$11,310	$2,828	$14,138
			$2,441,653	$398,260	$2,839,913

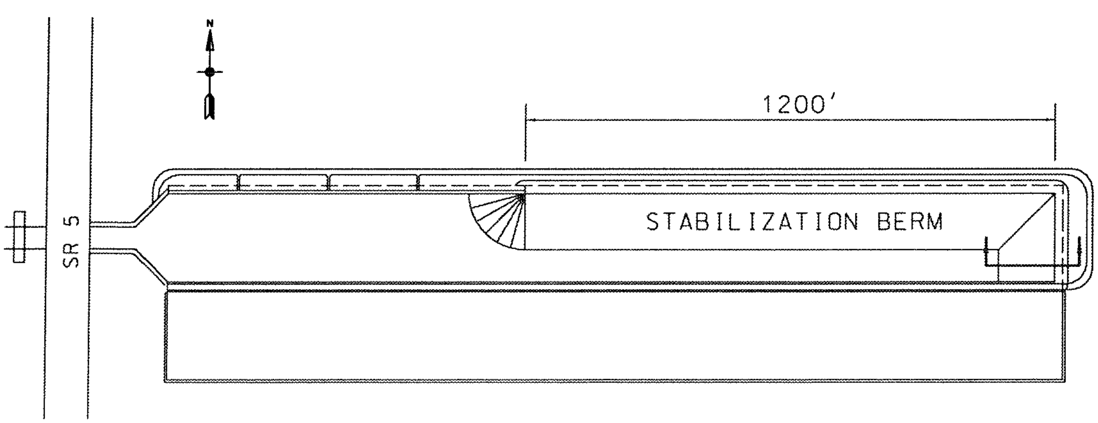
SAND SLOPE STABILIZATION OPTION

REACH D - SOUTH 100 L.F. OF EAST END WALL



SECTION

GENERAL LAYOUT SLOPE STABILIZATION OPTION



NOTES:

1. FILL MATERIAL TO BE CLEAN, DREDGED SAND.
2. SOIL REMOVED SHALL BE DISPOSED OF IN A CONFINED DISPOSAL FACILITY.
3. REINFORCED CONCRETE REMOVED TO BE DISPOSED OF.
4. RIPRAP SLOPE BETWEEN -3 LWD AND +8 LWD.

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UNION SHIP CANAL
WALL STABILIZATION OPTIONS

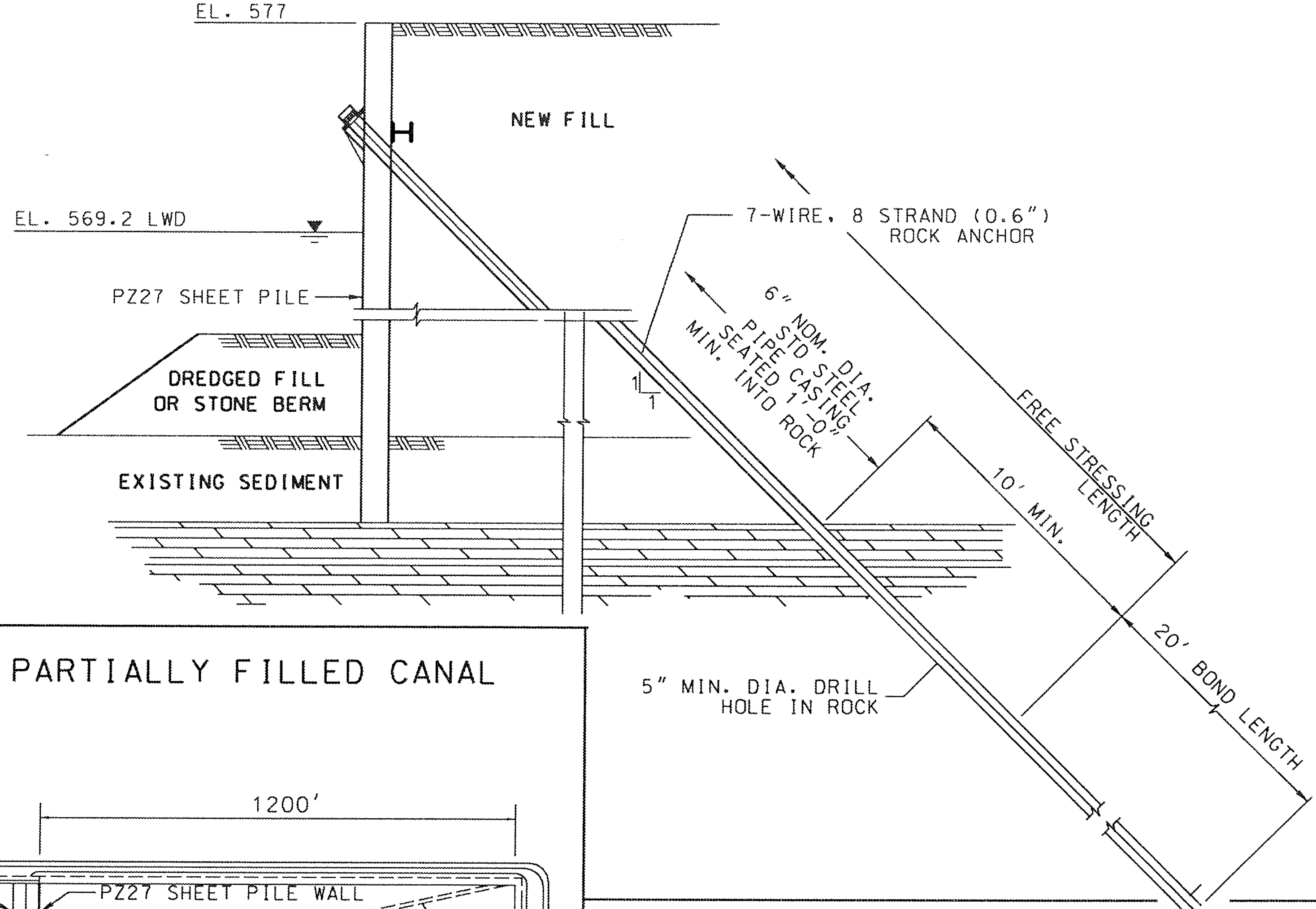
DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

UNION SHIP CANAL, BUFFALO, NEW YORK
PLANNING ASSISTANCE TO STATES

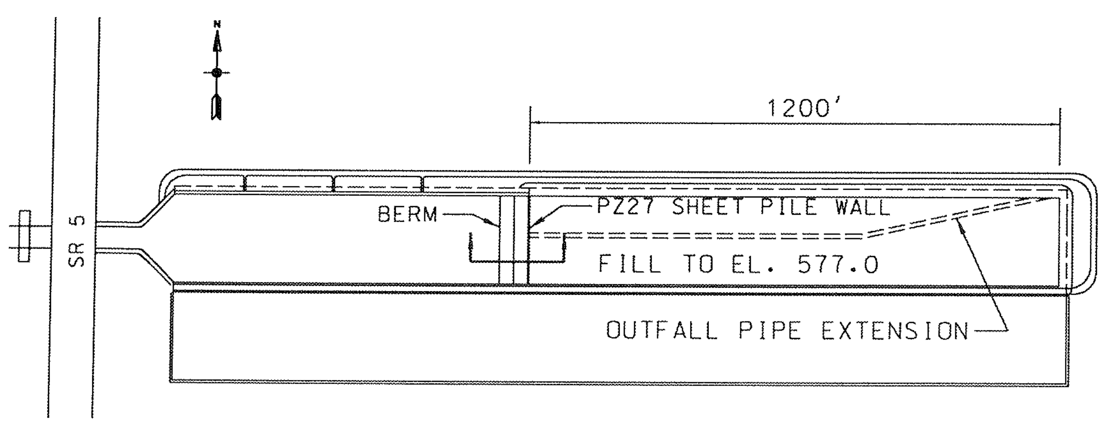
COST ESTIMATE
STONE FILL TO ELEVATION 577

FEATURE	QUANTITY	UNIT OF MEASURE	CONTRACT	CONTINGENCY	TOTAL COST
REMOVALS			$114,314	$28,578	$142,892
CLEARING & GRUBBING	3	ACRE	$20,560	$5,140	$25,701
STONE FILL	392400	TON	$5,886,000	$882,900	$6,768,900
PIPE EXTENSION	1	EACH	$291,045	$72,761	$363,806
WALK-WAY	1	EACH	$336,001	$84,000	$420,002
TOPSOIL & TURF			$208,333	$52,083	$260,416
TOPSOIL & SEED			$117,718	$29,430	$147,148
			$6,973,971	$1,154,892	$8,128,865

PARTIALLY FILLED CANAL WITH STEEL SHEET PILE WALL AND ROCK ANCHORS



GENERAL LAYOUT PARTIALLY FILLED CANAL



UNION SHIP CANAL

UNION SHIP CANAL
WALL STABILIZATION OPTIONS

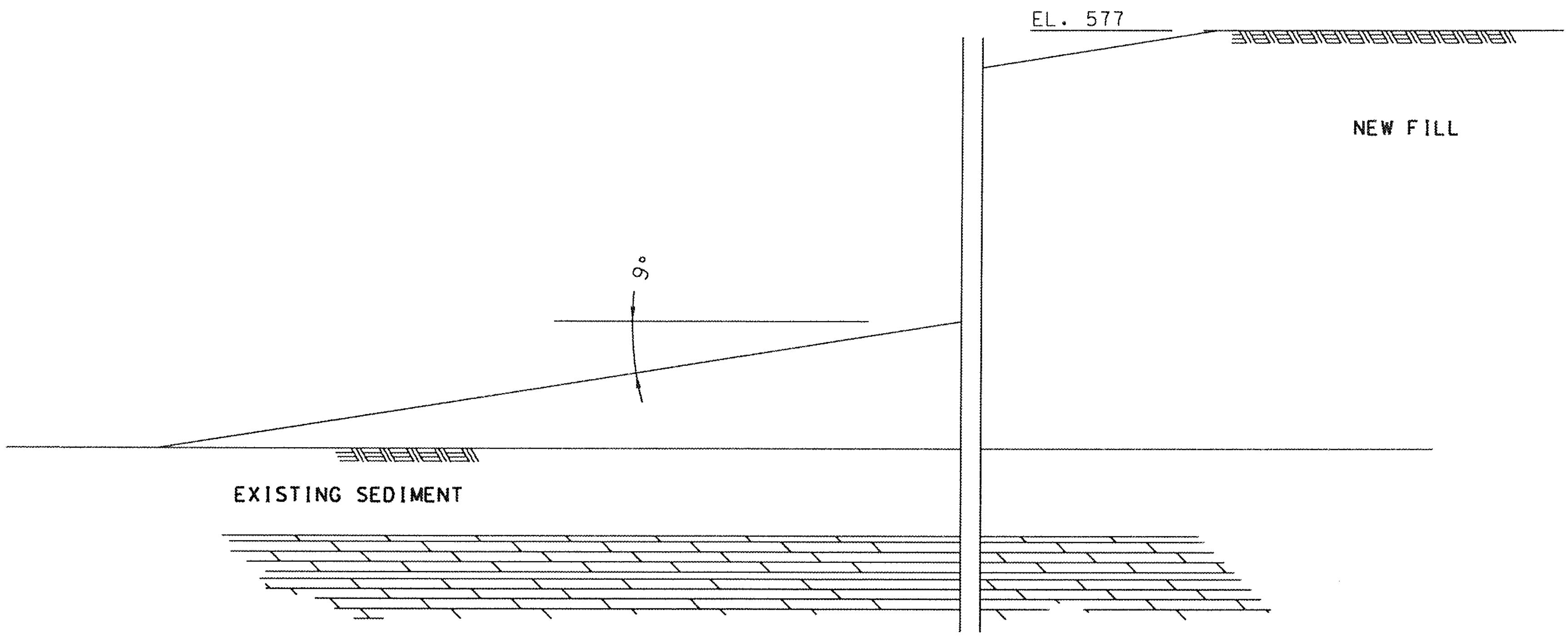
DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

UNION SHIP CANAL, BUFFALO, NEW YORK
PLANNING ASSISTANCE TO STATES

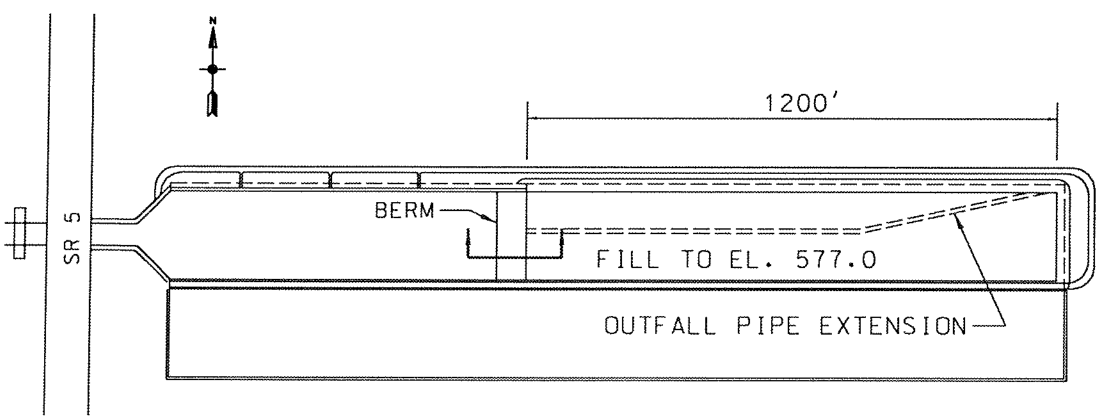
COST ESTIMATE
STEEL SHEET PILE ACROSS CANAL

FEATURE	QUANTITY	UNIT OF MEASURE	CONTRACT	CONTINGENCY	TOTAL COST
REMOVALS			$114,314	$28,578	$142,892
CLEARING & GRUBBING	3	ACRE	$20,560	$5,140	$25,701
STEEL SHEET PILE SYS			$472,750	$103,950	$576,700
FILL	267000	CY	$2,136,000	$534,000	$2,670,000
PIPE EXTENSION	1	EACH	$291,045	$72,761	$363,806
WALK-WAY	1	EACH	$336,001	$84,000	$420,001
TOPSOIL & SEED			$117,718	$29,430	$147,148
STONE BERM	850	TON	$38,250	$9,563	$47,813
			$3,526,638	$867,422	$4,394,061

PARTIALLY FILLED CANAL WITH BERM



GENERAL LAYOUT PARTIALLY FILLED CANAL



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WALL STABILIZATION OPTIONS

DRAWING NOT TO SCALE
ALL ELEVATIONS SHOWN REFERENCE IGLD 1985

US Army Corps of Engineers
Buffalo District

Union Ship Canal
Buffalo, New York

Sediment Analysis

Appendix E

US Army Corps of Engineers
Buffalo District

UNION SHIP CANAL
ERIE COUNTY; BUFFALO, NEW YORK

SEDIMENT EVALUATION

1. INTRODUCTION

The purpose of this report is to determine the suitability of sediments taken from the Union Ship Canal (USC) for open-lake disposal. The USC is located within Erie County, Buffalo, New York, and is situated along the eastern shore of Lake Erie off the southern end of Buffalo Harbor's Outer Harbor, approximately 3.5 miles southeast of the mouth of the Buffalo River (Figure 1).

2. DESCRIPTION OF SEDIMENT DATA

Sediment data for this report was obtained from a January 2000 sampling report prepared by PADIA Environmental, Inc. for the U.S. Army Corps of Engineers, Buffalo District (USACE). Eleven samples were taken from the USC during this sampling event. Figure 2 shows the locations where each sample was collected. These samples were subjected to the following analyses: Polychlorinated Biphenyls (PCBs), Polynuclear Aromatic Hydrocarbons (PAHs), Metals, Oil and Grease, and Ammonia Nitrogen.

3. METHODS OF EVALUATION

A Tier I Analysis was performed in accordance with guidance contained in the Great Lakes Dredged Material Testing and Evaluation Manual (USEPA/USACE 1998). In order to determine the suitability of the sediments for open-lake disposal, the results of the sediment testing analyses from each site were compared to the results obtained from an open-lake reference area. The open-lake reference area results are used to represent background contaminant concentrations in Lake Erie sediments. If the contaminant level measured in the USC sediment is less than or relatively comparable to that in the open-lake reference area sediment, it is considered suitable for open-lake disposal. The results for the open-lake reference area were taken from a 1996 sampling report prepared for the Buffalo District by Engineering and Environment, Inc. Two samples were taken from an open-lake reference area during this sampling event for the Buffalo Harbor area. The range of values measured from these two sites was determined for each parameter. The USC sediment results were then compared to this reference area range for each parameter. If the value of the parameter obtained from the USC sediment was within or comparable to the reference area range, it passed Tier I analysis and was

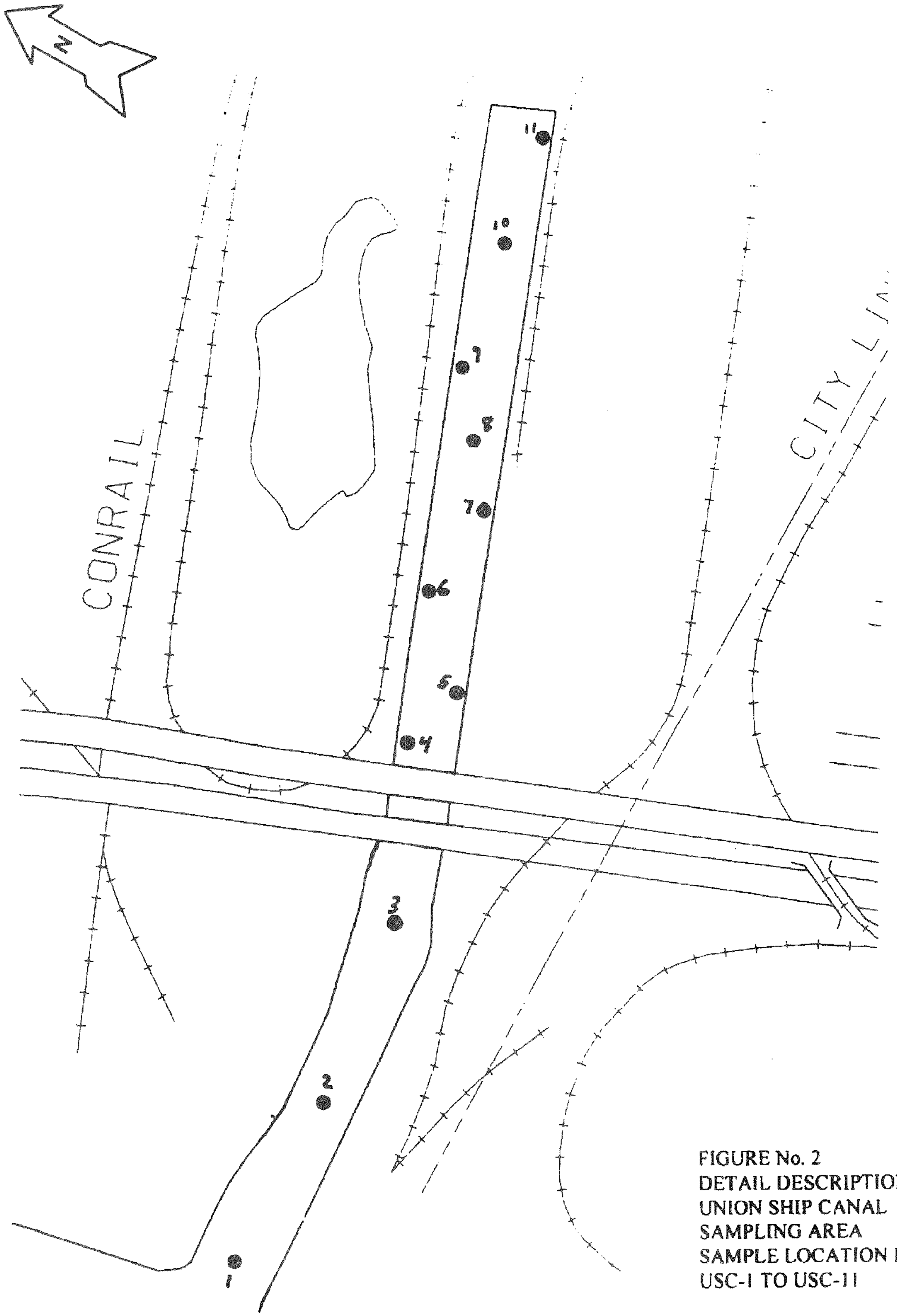

FIGURE No. 2
DETAIL DESCRIPTION OF
UNION SHIP CANAL
SAMPLING AREA
SAMPLE LOCATION POINTS
USC-1 TO USC-11

not subjected to any further evaluation. A site passed Tier I analysis for PAHs if the Total PAHs value for that site was within the reference area range, even if it had individual PAH compounds that exceeded the reference area range. PAH parameters that exceeded the reference area range were subjected to additional analysis. These parameters are indicated in Tables 1 and 2 with **bold** lettering. Note that any results presented with a "<" symbol were undetected during laboratory testing.

Sites that failed Tier I analysis for PAHs were evaluated using the Toxicity Equivalent (TEQ) model. This is a model that estimates the cumulative effect of the carcinogenic PAH compounds based on their relative toxicity. Toxicity Equivalent Factors (TEFs) were developed for the seven carcinogenic PAHs as shown below:

Benzo[a]pyrene -	1.0
Benzo[a]anthracene -	0.1
Benzo[b]fluoranthene -	0.1
Benzo[k]fluoranthene -	0.01
Chrysene -	0.001
Dibenzo[a,h]anthracene -	1.0
Indeno[1,2,3-cd]pyrene -	0.1

The results of each of the carcinogenic parameters were multiplied by their respective TEF, and the sum of the seven resultant values represents the TEQ at that site. The TEQs calculated from the USC sediment sites were compared to those calculated from the reference area. If the TEQ of the USC sediment was less than that calculated for the reference area, it was considered suitable for open-lake disposal. The other PAH parameters are not carcinogenic and are therefore considered to be of little toxicological significance at relatively low levels.

4. RESULTS

After conducting Tier I analysis, and, where necessary, applying the TEQ model, a determination was made for each site regarding suitability for open-lake disposal. To be suitable for open-lake disposal, a site must not have any contaminant parameters that did not pass Tier I analysis and/or the TEQ model. The following is a narrative description of the results of evaluations conducted on the USC sediments.

a. POLYNUCLEAR AROMATIC HYDROCARBONS (PAHs)

Every site had multiple PAH compounds that exceeded the reference area range. Table 1 lists the PAH compounds and their results at each site. The Total PAH level at each site also exceeded the Total PAH range measured at the reference area. The TEQ model was therefore applied to each site, and again, the TEQs calculated from the USC sediments exceeded those calculated from the reference area.

b. POLYCHLORINATED BIPHENYLS (PCBs)

The only PCBs that were detected in the USC sediments were Aroclor 1254 and Aroclor 1260 (Table 1). However, no PCBs were detected in the reference area, so any presence of PCBs in the USC sediments exceeds that found in the reference area.

c. METALS

Every metal was detected at every site in the USC sediments in levels greater than those in the reference area (Table 2). Lead and Zinc were detected at particularly high levels.

d. INORGANIC CONSTITUENTS

Ammonia Nitrogen and Oil and Grease were the inorganic compounds evaluated for the USC sediments. Both compounds were detected at all sites in levels greater than those in the reference area (Table 2).

5. HAZARDOUS WASTE CONSIDERATIONS

Because of the high levels of lead and zinc, 40 CFR 261 was referenced to determine if the sediments from the USC had the potential to be identified as hazardous waste under the Resource Conservation and Recovery Act (RCRA), particularly for the characteristic of toxicity. A solid waste exhibits the characteristic of toxicity if, using the Toxicity Characteristic Leaching Procedure (TCLP) - test Method 1311, the waste extract contains concentrations of contaminants greater than those listed in 40 CFR 261.24(b). Since the TCLP test was not conducted on the sediments, the "20 Dilution Rule" was used. The "20 Dilution Rule" can be used to provide a conservative estimate of TCLP results based on a total analysis. This estimation method takes into account that a 20-fold dilution is done during the TCLP. Consequently, a total analysis for lead in excess of 100 ppm would be conservatively estimated to exceed the regulatory limit of 5 ppm for lead listed in 40 CFR 261.24(b). Zinc is not listed in 40 CFR 261.24(b), therefore this procedure was not conducted on the zinc results.

USC sites 3 – 11 had results that exceeded the 100 ppm level for lead and consequently have a potential to exceed the hazardous waste regulatory limit for lead. Consequently, prior to a disposal decision, TCLP testing would be required to determine with certainty whether the sediments exhibited the characteristic of toxicity for lead (USEPA Hazardous Waste Number D008). If the TCLP test determines that the sediments are indeed a hazardous waste, a specially permitted landfill would be required for disposal of the sediments, and all applicable transportation and handling requirements must be considered.

6. CONCLUSIONS

All of the sediments from the USC have been determined to be unsuitable for open-lake disposal. The contaminant levels in these sediments exceed the levels that are found in the open-lake sediments. Therefore these sediments cannot be placed in the open-lake. There are three general options for sediments determined to be unsuitable for open-lake disposal based on Tier I analysis:

a. Conduct biological testing on the sediments to assess their potential affect on aquatic biota to determine their suitability for open-lake disposal. This option is considered to be infeasible for the Union Ship Canal. The level of contaminants found in these sediments is such that the chance of yielding biological testing results similar to those in the open-lake reference area is minimal. Subjecting the sediments to biological testing is not anticipated to yield favorable results, and therefore would not justify the money and effort required to undertake the testing.

b. Disposal of the sediments in a confined disposal facility (CDF). Additional testing of the sediment, including TCLP, and analysis of the results would be required to determine if this alternative is feasible. Buffalo Harbor's CDF #4 is located less than 0.5-mile from the canal, and would be readily accessible if space is available, the sediment is determined to be suitable and authorization to use the CDF can be obtained.

c. Dewatering and subsequent upland disposal in a municipal landfill. This would also be a feasible alternative, providing a viable landfill can be located for the sediments. However, the water removed in the dewatering process may have to be treated for disposal. Additionally, as previously discussed, TCLP testing would be required to determine with certainty whether the sediments exhibited the characteristic of toxicity for lead (USEPA Hazardous Waste Number D008). If the TCLP test determines that the sediments are indeed a hazardous waste, a specially permitted landfill would be required for the disposal of the sediments, and all applicable transportation and handling requirements must be considered.

REFERENCES

Engineering and Environment, Inc. 1996. *Sediment Sampling for Chemical and Particle Size Analysis: Buffalo Harbor, New York.*

PADIA Environmental, Inc. 2000. *Final Report for Sediment Sampling and Chemical Analysis at the Union Ship Canal in Buffalo, New York.*

USEPA. 1998. *Title 40 Code of Federal Regulations, Subpart C – "Characteristics of Hazardous Waste", Section 261.20.*

USEPA/USACE. 1998. *Great Lakes Dredged Material Testing and Evaluation Manual – Final Draft.*